PROCEEDINGS

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1932

VOLUME 58 NUMBER 6

PROCEEDINGS

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OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 58 AUGUST, 1932 No. 6

CURRENT

TECHNICAL PAPERS

DISCUSSIONS

APPLICATIONS FOR ADMISSION AND TRANSFER

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MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

GEORGE WASHINGTON BRIDGE: GENERAL CONCEPTION AND DEVELOPMENT OF DESIGN

By O. H. Ammann,[1] M. Am. Soc. C. E.

Synopsis

The principal purpose of this paper is to review the developments which after many years of effort have led to the realization of the bridging of the Hudson River at New York, N. Y.; to outline the conditions and considerations which governed the planning of the bridge between Fort Washington and Fort Lee, known as George Washington Bridge; and to record the conception and development of its design in general, the organization, progress, and the cost of its construction.

In view of the magnitude and complexity of the project, this paper is to be supplemented by a series of papers which will deal, more in detail, with the various important phases of the project.

History, Legislation, and Financing

The bridging of the Hudson River at New York is one of those civic and engineering undertakings which for generations have attracted the ambitions and efforts of public-spirited men and of engineers, but the development of which to the point of success must, on account of their magnitude and the many ramifications, consume years and are attained only after a series of unsuccessful attempts.

Credit for the ultimate success, therefore, is due not only to those who are so fortunate as to accomplish the execution, but as much, or more, to those pioneers who by their vision and courage have pointed the way, and whose ideas and studies have prepared the ground for the final accomplishment.

As might be expected, the various attempts to bridge the Hudson, initiated by different interests and involving different conceptions, have produced a wealth of varied ideas regarding the location of crossings and their type and

Note.—Discussion of this paper will be closed in November, 1932, *Proceedings*.

[1] Chf. Engr., The Port of New York Authority, New York, N. Y.

capacity, not to speak of the many divergent opinions regarding questions of design.

While it is to be assumed that the projects which have thus far materialized owe their success to some outstanding merits, it is also undoubtedly true that favorable circumstances combined to effect their realization. The problems involved are so complex and so fraught with ramifications politically, financially, as well as technically, and subject to such rapidly changing conditions that it would be preposterous to assign to any particular project the virtue of, even passing, perfection and completeness.

In the fifty years since 1880, during which concrete efforts have been made to bridge the Hudson River, conditions which would influence any of the major questions involved in this problem, have changed radically.

Traffic in New York City has increased materially in volume and its center of gravity with that of population and business, has moved steadily north. In the early attempts, the demand for rail traffic was dominant. Crossings for vehicular traffic were scarcely ever considered. Tunnels were thought to be impracticable even for rail transportation. To-day, any bridge across the Hudson River at New York must be viewed primarily as a highway structure, only incidentally accommodating rail traffic, and it is in sharp competition with the tunnel.

More severe demands are made to-day by navigation interests for clearances under a bridge across the Hudson River, and it appears certain that the War Department will permit nothing less than 175 ft. in clear height.

The development of property and the congested conditions on either side of the river also impose such severe limitations upon the building of bridges as to exclude them, practically, for any location in the lower part of Manhattan.

There has also been a marked change in the attitude of the Governments, reflecting public opinion, with respect to the method of financing and carrying out such far-reaching public improvements as crossings over or under the Hudson River.

Joint financing and building by public agencies representing the two States and under State legislation sanctioned by the Federal Government have proved eminently successful and have gained public confidence.

The history of bridging the Hudson River is necessarily interwoven with the broader developments regarding transportation in and around the Port of New York, but in order to give the project dealt with in this paper its proper historic setting, it will suffice to review briefly the various previous attempts.

Projects of the New York and New Jersey Bridge Company and Investigations by the Federal Government.—As early as 1868, the State of New Jersey passed an Act authorizing the New York and New Jersey Bridge Company to build a bridge across the Hudson River at a suitable point north of the southerly line of the Township of Union. The Act permitted a bridge with one or two piers in the river between the bulkhead lines with clear openings of not less than 1 000 ft. and a clear height of 130 ft. at the middle of the river.

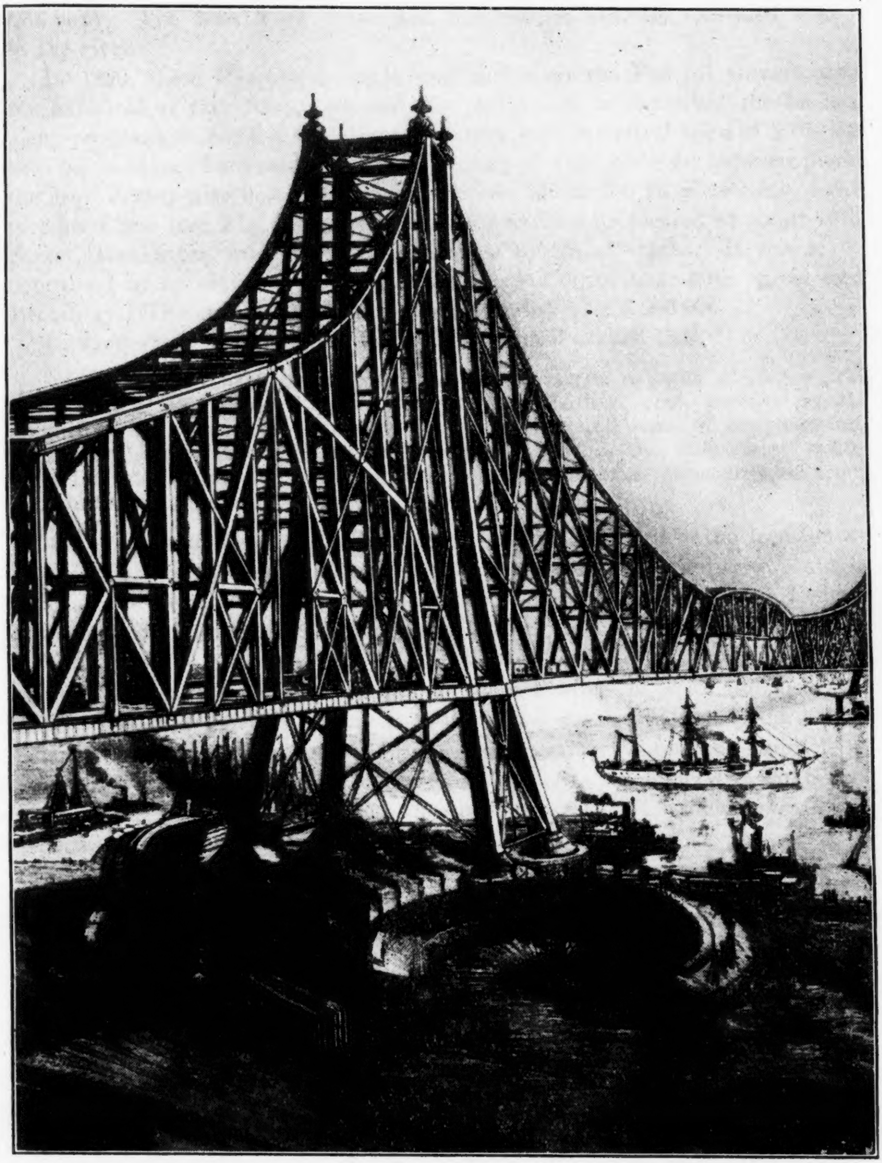

FIG. 1.—CANTILEVER BRIDGE NEAR 70TH STREET, MANHATTAN, PROPOSED IN 1893.

In accordance with its provisions the Act did not become effective until, in 1890, New York State passed concurrent legislation giving a charter to a similar company and providing for its consolidation with the New Jersey Company. The New York State Act, however, specifically excluded a pier in the river.

In 1890, these Companies made application to the Federal Government for approval of the State. Acts and for permission to construct the bridge. They proposed to build a cantilever structure with a central span of 2 300 ft. between centers of towers, or a clear opening of only 2 000 ft. between piers, the New Jersey pier to be located in the river about 900 ft. riverward of the pier-head line (see Fig. 1). The bridge was tentatively located at about 70th Street, Manhattan, and designed to carry six railroad tracks. It was to be connected by an elevated approach with a union depot near 40th Street and Broadway. The cost of the bridge was estimated at $22 000 000.

In support of this plan, the Bridge Companies[2] argued that,

"A suspension bridge spanning the North River without a pier would involve such elements of uncertainty as regards first cost, novelty in its magnitude as a hitherto untried engineering feat, and time of construction, to say nothing of the well-founded prejudice against the 'suspension' principle for railroad purposes, as would render the enterprise impracticable from a financial standpoint."

On account of much opposition which had arisen against the location of any pier in the river and the controversy relative to the feasibility of a single span across the river, an Act was finally passed and approved by the Federal Government in 1894, authorizing the New York and New Jersey Companies to build a bridge between 59th and 60th Street, New York City, but provided therein the plans must be approved by the Secretary of War and also that the President of the United States should appoint a Board, consisting of five competent, disinterested expert bridge engineers, of whom one must be a member of the United States Corps of Engineers, to recommend to the Secretary of War "what length of span, not less than 2 000 ft., would be safe and practicable for a railroad bridge." [2]

Accordingly, a Board composed of the following engineers was appointed by President Cleveland: William H. Burr, M. Am. Soc. C. E. and the late George S. Morison, Past-President, Am. Soc. C. E., and Charles W. Raymond, L. G. L. Bouscaren, and Theodore Cooper, Members, Am. Soc. C. E.

The Secretary of War had previously appointed a Board of Officers of the United States Corps of Engineers with instructions to "investigate and report their conclusions as to the maximum length of span practicable for suspension bridges and consistent with an amount of traffic probably sufficient to warrant the expense of construction." This Board was composed of Col. Edward Burr, U. S. A. (*Retired*) (then Captain, Corps of Engineers, U. S. A.), M. Am. Soc. C. E., and the late Brig.-Gen. William H. Bixby, U. S. A. (*Retired*), M. Am. Soc. C. E., (then, also, Captain, Corps of Engineers, U. S. A.), and the late Maj. Charles W. Raymond.

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[2] See Senate Ex. Doc. No. 12, 53d Cong., 3d Session.

The reports of both of these Boards,[2] which have become classic documents, furnished valuable and exhaustive information and definitely disposed of the question of the feasibility of a single span across the Hudson River in New York and the adaptability and economy of the suspension type for long spans. The reports also contain valuable and interesting statements and studies by Gustav Lindenthal, Hon. M. Am. Soc. C. E., and the late Charles Macdonald, Past-President, Am. Soc. C. E., and Wilhelm Hildenbrand, M. Am. Soc. C. E., as well as a theory of the stiffening girder by Professor J. Melan.

The report of the Board of Engineers closed with the following recommendation:

"The only subject referred to your Board is 'to recommend what length of span not less than 2 000 ft. would be safe and practicable for a railroad bridge to be constructed over the Hudson River between Fifty-ninth and Sixty-ninth Streets.' A single span from pier-head to pier-head, built on either the cantilever or suspension principle, would be safe. The estimated cost of the 3 100 foot clear-span cantilever being about twice that of the shorter span, your Board consider themselves justified in pronouncing it impracticable on financial grounds. As the cost of the single span suspension bridge is at most (not more than) one-third greater than that of the 2 000 ft. cantilever, your Board are unable to say that such greater cost is enough to render the suspension bridge impracticable.

"The Board have reached this conclusion after careful study, and they have thought it best to give the full course of reasoning which they have followed. They feel that the contingency attending the construction of the deep-river foundation of the cantilever bridge, even waiving the absolute necessity of carrying this foundation to rock, is enough to balance a part of the greater cost of the suspension bridge.

"The conclusion of this Board is that of a Board of Bridge Engineers acting in a professional capacity. While from such professional view they must pronounce the suspension bridge practicable, they do not in this conclusion give an opinion on the financial practicability and merit of either plan."

The conclusions of the Board of Engineer Officers as endorsed by the then Chief of Engineers, U. S. Army, Gen. Thomas L. Casey, were as follows:

"The final plans for work of such magnitude would only be adopted after the most extended theoretical and experimental investigations, and the estimated cost would undoubtedly be much reduced by such studies. Assuming the most favorable location and the most competent engineering management, the Board believe that $23 000 000 is a reasonable estimate for a six-track railroad suspension bridge 3 200 feet long, and they consider the amount of traffic which such a bridge would accommodate sufficient to warrant the expense of construction. They believe, however, that the bridge should be so constructed that its capacity can be readily increased, and with the suspension system this can be provided for by giving suitable dimensions to the towers and anchorages."

Briefly, the conclusions are to the effect that both the 2 000-ft. cantilever bridge, which requires a pier in the river and the 3 200-ft. suspension bridge without a center pier, are safe and not impracticable as to cost, and that the latter type, in spite of its greater span, would not cost materially more. The estimates of various studies ranged between about $25 000 000 and $35 000 000, depending upon location and capacity.

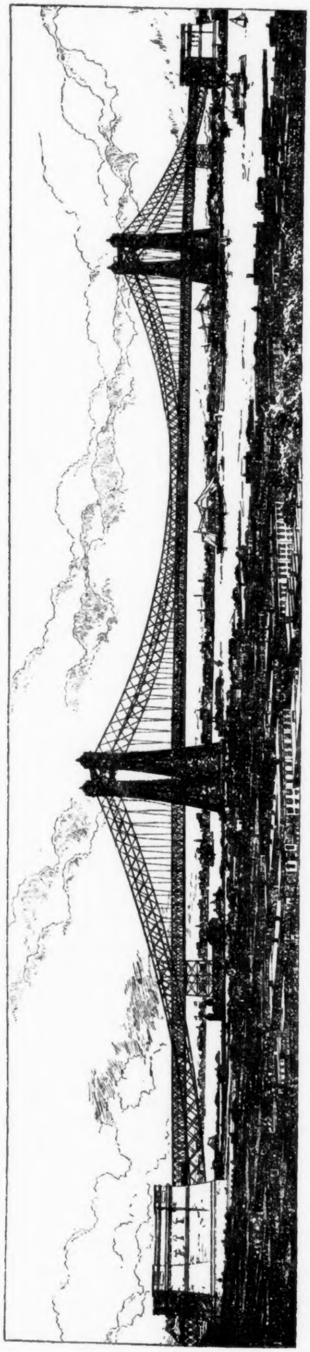

FIG. 2.—SUSPENSION BRIDGE NEAR 23D STREET, MANHATTAN, DESIGNED IN 1899.

As a result of these findings, the Secretary of War disapproved of the proposed cantilever structure with a pier in the river.

In the meantime the New York and New Jersey Bridge Company had changed its plans and thereafter proposed a six-track suspension bridge in the vicinity of 57th Street with a single river span at an estimated cost of \$25 000 000 for the bridge proper. A number of wash borings were made for the Company by the late Charles B. Brush M. Am. Soc. C. E., at both the 71st and 59th Street locations, but no construction work was ever undertaken.

The Project of the North River Bridge Company.—At a meeting of the Society in January, 1888, Mr. Lindenthal outlined his carefully studied plan for a railroad suspension bridge across the Hudson River. It was a remarkably bold and well conceived plan, calling for the first time for a single span across the river, 2 850 ft. in length, and two side spans of 1 500 ft. each, or a total length between anchorages of 5 850 ft. (Fig. 2). It provided for six railroad tracks to be carried by four 48-in. cables, braced in pairs to form rigid suspended trusses, slightly cradled (Fig. 3). The cables were to be suspended from two pairs of octagonal-shaped steel towers, 525 ft. high, of massive proportions. Subsequently, the plan was modified to provide for as many as fourteen tracks and a promenade, and the span was increased to 3 100 ft.[3].

The bridge proper was estimated to cost \$16 000 000 and, with terminal facilities, \$40 000 000. A location in the vicinity of 10th Street, New York, was at first selected, but this was later changed to the vicinity of 23d Street, opposite Hoboken, N. J.

The publication of the plans caused widespread and favorable comment, and, in 1890, the Federal Government granted a charter to the North River Bridge Company, by passing an Act authorizing Mr. Lindenthal and his associates (among whom were such other well-known engineers as Samuel Rea, Hon. M. Am. Soc. C. E., Henry Flad, M. Am. Soc. C. E., and Mr. F. W. Roebling, none of whom is now living), to construct a bridge "for at least six railroad tracks, with capacity for four additional tracks for future enlargement, and with a single span across the river between pier-head lines." The charter also provided for the building of the necessary approaches and terminal facilities. The plans for the bridge were approved by the Secretary of War, in December, 1891, who fixed the clear height at the center at 150 ft.

An attempt to build the bridge at that time failed, due evidently to the financial stringency in 1893 and the consequent inability of the railroad companies to co-operate, this being essential for the success of a railroad terminal improvement of such magnitude.

Another effort was made in 1900, when the Pennsylvania Railroad Company invited the other railroads terminating on the west shore of the Hudson River to join in the building of the bridge; but these companies did not avail themselves of the opportunity and the Pennsylvania Railroad Company, having become convinced of the feasibility of electric railroad traction, decided to enter Manhattan by two single-track tunnels near 33d Street.[4]

[3] *Scientific American*, May 23, 1891, p. 319.

[4] *Transactions*, Am. Soc. C. E., Vols. LXVIII and LXIX (1910).

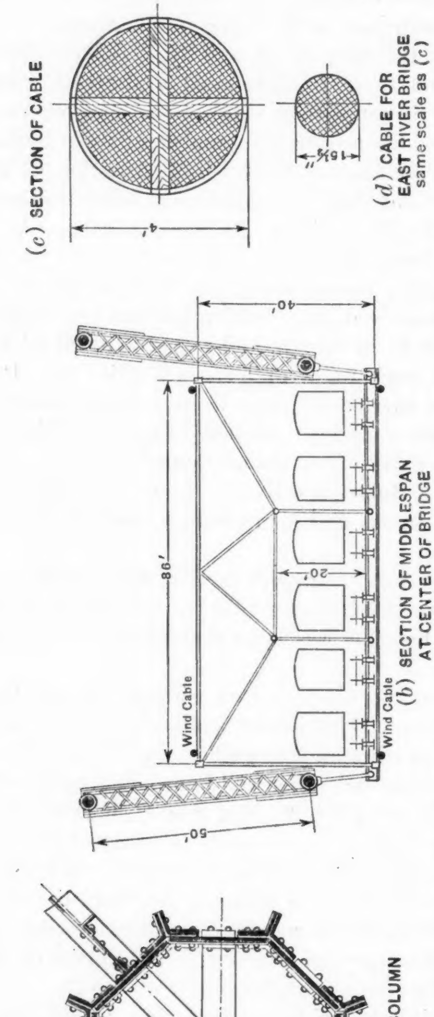

FIG. 3.—SECTIONS FOR SUSPENSION BRIDGE NEAR 23D STREET, MANHATTAN, DESIGNED IN 1899.

Furthermore, having been encouraged by the example of the Pennsylvania Railroad, and in co-operation with the latter, the Hudson and Manhattan Railroad Company, in 1910, completed its two pairs of tubes connecting three of the passenger stations on the New Jersey side with a down-town terminal and with points along Sixth Avenue, as far north as 33d Street.

The successful completion of these tubes for electric rail traffic constituted a setback to the possibilities of a railroad bridge in that they established beyond doubt the feasibility of tunnels for rail traffic and obviated the necessity for additional crossings for the time being.

The phenomenal growth of vehicular traffic after the World War and the renewed efforts, more particularly on the part of the New York and New Jersey Port and Harbor Development Commission (later succeeded by the Port of New York Authority), to improve rail terminal facilities in New York, gave new encouragement to the sponsors of a bridge to revive the North River Bridge Company's plans.

The plans for both the bridge proper and the terminal facilities were completely revised by Mr. Lindenthal. The location had previously, upon permis-

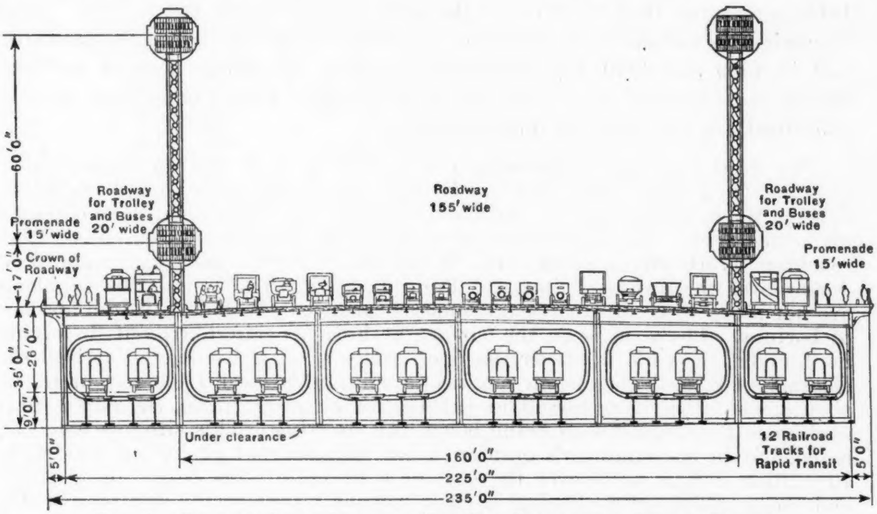

FIG. 4.—TYPICAL CROSS-SECTION; EYE-BAR SUSPENSION BRIDGE DESIGN, SUBMITTED IN 1923.

sion by the Federal Government, been removed to the vicinity of 57th Street, Manhattan. The bridge proper was re-designed for a capacity of twelve railroad and rapid transit tracks, twenty lanes of vehicular and bus traffic, and two 15-ft. promenades, with a total width of the double deck floor of 235 ft. (Fig. 4). The suspended system with a central span of 3 240 ft. remained essentially the same, except in its proportions and the substitution of eye-bar chains in the place of wire cables. The towers were designed as rigid steel frames to be enveloped by an independent shell of masonry of massive appearance. The bridge proper and highway approaches were estimated to cost approximately $180 000 000.

With the assistance of Francis Lee Stuart, Past-President, Am. Soc. C. E., and in consultation with the engineers of the various railroad companies, Mr. Lindenthal developed an elaborate plan for freight and passenger terminal facilities. The plan was submitted to the Port of New York Authority in 1921, but the latter, while recognizing the merits of the bridge as a highway proposition, could not see its way clear to adopt it as a feature of its comprehensive plan in the solution of the freight terminal problem.[5]

In 1923, the North River Bridge Company submitted the revised plans to the War Department, proposing a clear height under the bridge at the center of 175 ft. After lengthy consideration, the War Department on June 9, 1931, rendered a decision that the clear height should not be less than 200 ft. at the center and 185 ft. at the pier-head lines.

The Interstate Bridge and Tunnel Commissions of New York and New Jersey and the Holland Vehicular Tunnel.—In 1906, the Governors of New York and New Jersey, acting under laws passed in that year by the respective Legislatures, appointed Commissions, known as the Interstate Bridge Commissions, for the purpose of considering the construction of one or more bridges over the Hudson River at the joint expense of the two States. These Commissions collaborated in making a careful study of various possible sites and in 1909 and 1910, reported favorably upon the bridge located at 179th Street, opposite Fort Lee. The report of the New York Commission of 1910 contained the following recommendations:

"From the purely engineering point of view it is the most economical crossing from Manhattan over the Hudson River that it is possible to select, it being the narrowest part of the river, with comparatively small land damages on either side. The approaches over land are short, that from New York reaching 179th Street over Fort Washington Park, and that from New Jersey over the proposed limits of Palisade Park. The foundation problems are not likely to be of great magnitude as far as can be judged in the absence of borings. The rock is on the surface at Fort Washington point, involving no foundation work whatever, beyond levelling off the same. Further, the channel span need not, in our engineer's opinion, be over 1 400 feet or thereabouts, which will give abundant passage for all river traffic, the north limit anchorage for large vessels being below this crossing. This site has not been bored, but in our engineer's opinion, from the apparent geological condition, 10 million dollars will cover the cost of a bridge at this point for highway and speed trolley service, being in their opinion one-third the cost of a bridge lower down the river."

Subsequently, borings were undertaken at the 57th, 110th, and 179th Street locations. The assumption that piers could be placed in the river at the 179th Street site, and that the foundation problems would not be of great magnitude, was not supported by the results of these borings and, with the prospect that a bridge at that point would require a single span across the river, as elsewhere, the engineers of the Commissions, the late A. P. Boller and Henry W. Hodge, Members, Am. Soc. C. E., reported in 1911, as follows:

"The borings conclusively prove that there are no practicable foundation conditions outside of the pier head lines, forcing the inevitable conclusions that

[5] See statement by Eugenius H. Outerbridge, Chairman, Port of New York Authority, to the Advisory Council, December 8, 1921.

any bridge contemplated over the Hudson River within the limits of the City of New York must have at least a single span over the river between the pier head lines. Inasmuch as the distance between pier head lines is substantially the same at any proposed crossing between 57th Street and 179th Street, the constructive cost of a bridge at any site will practically be the same; such being the case your engineer is of the opinion that the final determination of the bridge location should be guided by the line of greatest natural travel and public convenience. While the cost of real estate and abutting damages will vary according to location, still it is believed that the needs of the community served should be controlling within reasonable limits. All things considered, it is the firm opinion of your engineer that a bridge located in the neighborhood of 59th Street will best conform to the needs of population density and requirements on both sides of the river, and such location is recommended for adoption."

Thereafter, the plan for a bridge at 179th Street seems to have been abandoned by the Interstate Bridge Commissions, and, in 1913, they recommended a bridge near 59th Street, a design for which had been prepared by Messrs. Boller and Hodge, and H. C. Baird, M. Am. Soc. C. E. At the same time, the Commissions reported favorably upon a vehicular tunnel at Canal Street, which latter project had been investigated and recommended by Messrs. Jacobs and Davies, Consulting Engineers, as being feasible and economical.

It is quite evident that by this time the need for more adequate crossings for vehicular traffic had come to the foreground, and furnished new possibilities for bridging and tunneling the Hudson River.

The beginning of the World War delayed the undertaking of either project, and only in 1919, when the necessity for a crossing became urgent as a result of the rapidly growing vehicular traffic and activities arising out of the war, the States of New York and New Jersey entered into a treaty for the construction of a vehicular tunnel by and through the New York Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission (successors of the aforementioned Bridge Commissions), the outcome of which was the successful completion, in 1927, of the Holland Tunnel between Canal Street, Manhattan and Jersey City, N. J.

Preference to the tunnel project over the bridge at 57th Street was evidently given partly on account of the location of the tunnel down town, where a crossing could be of more immediate relief to vehicular traffic; and partly due to its apparent lower cost, the tunnel having been estimated in 1913 at \$11 000 000 and the bridge at 57th Street at \$42 000 000.

The design of the bridge by Messrs. Boller, Hodge, and Baird contemplated a capacity of a single deck, 204 ft. wide, for eight rapid transit and trolley tracks, two roadways, each 36 ft. wide, and two sidewalks, each 8 ft. wide. The suspended truss type of bridge was selected, each of four trusses consisting of a main eye-bar cable stiffened by a secondary cable and the connecting web members. The central span was assumed at 2 880 ft. center to center of towers and the clear height over the river at 170 ft.

The Port of New York Authority and the Financing of the George Washington Bridge.—Realizing the urgent need for additional crossings between the two States, as brought about by the phenomenal growth of vehi-

cular traffic, Governor Alfred E. Smith, of New York State, and Governor George S. Silzer, of New Jersey, on August 5, 1923, issued a joint proclamation, in which they stated, in part:

"One of the results of the conference between the two Governors was that we favor the construction at the earliest possible moment of additional vehicular tunnels or bridges between the State of New York and the State of New Jersey to be determined upon, constructed and financed by the Port of New York Authority, and we stand ready to recommend to the Legislatures the passage of any additional legislation that will be helpful towards the accomplishment of this result."

On December 5, 1923, a public hearing on this subject was held by the Port Authority. The sentiment expressed at this hearing was almost unanimous in favor of the building of interstate vehicular tunnels or bridges by the Port of New York Authority and, likewise, in favor of two or more vehicular tunnels, and a bridge at some point north of 128th Street, Manhattan. There was substantial approval for a highway bridge at a location about 178th Street, Manhattan.

In its report[6] to the Governors of the two States in December, 1923, the Port Authority recommended that preliminary engineering and traffic studies and plans should be promptly undertaken relating to such crossings.

Substantial weight to the proposal for a bridge at 179th Street was given by a report to the Port Authority of the Committee on Plan of New York and Its Environs of the Russell Sage Foundation. In a subsequent publication[7] that Committee outlined a plan for the bridge in which a central span of 2 700 ft., with a pier approximately 400 ft. beyond the westerly pier-head line, was tentatively assumed.

In a communication to the Port Authority in December, 1923, Governor Silzer, of New Jersey, transmitted to that body for its consideration the study of a plan for a bridge at 179th Street which had been submitted to him by the writer and which, based upon a carefully studied design (Fig. 5), and estimates of cost and revenue, indicated the financial feasibility of the project. In its essential features the writer's tentative plan substantially agrees with the design eventually adopted for execution. It provided for a single span of 3 400 ft. across the river, with piers back of the pier-head lines, a capacity for eight lanes of vehicular traffic, two sidewalks, and four rapid transit tracks, and a clear height of 200 ft. above the water. (See Fig. 6.) The plan was presented by the writer at the Annual Meeting of the Connecticut Society of Civil Engineers on February 19, 1924.[8]

In 1925, the States of New York and New Jersey passed legislation authorizing and empowering the Port of New York Authority, in partial effectuation of the comprehensive plan for the development of the Port of New York, to construct, operate, and maintain a bridge across the Hudson River, from points between 170th Street and 185th Street, Manhattan, and points approximately opposite thereto in Fort Lee, N. J.

[6] Annual Rept. of The Port of New York Authority, January, 1924, p. 43.

[7] "Some Preliminary Suggestions for the Relief of Highway Congestion in New York," 1925.

[8] *Proceedings*, Connecticut Soc. of Civ. Engrs., 1924.

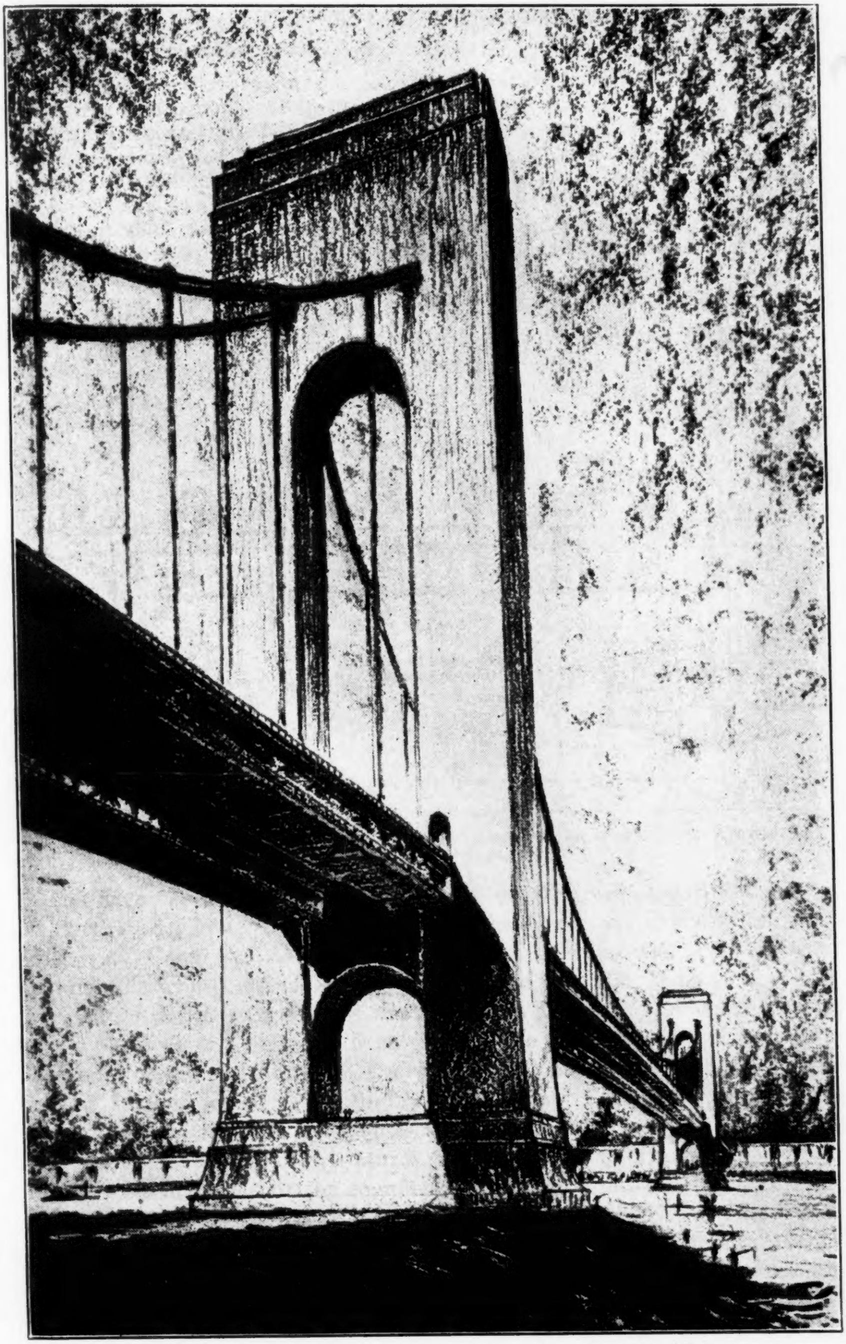

FIG. 5.—SUSPENSION BRIDGE PROPOSED FOR 179TH STREET, MANHATTAN, IN DECEMBER, 1923.

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In passing this legislation, the Legislatures of the two States fully recognized the fact, as stated in the Acts,[9] that,

"The construction, maintenance and operation of said bridge is in all respects for the benefit of the people of the two States, for the increase of their commerce and prosperity, and for the improvement of their health and living conditions, and the Port Authority shall be regarded as performing a governmental function in undertaking the said construction, maintenance, and operation and in carrying out the provisions of law relating to the said bridge and shall be required to pay no taxes or assessments upon any of the property acquired by it for the construction, operation, and maintenance of such bridge."

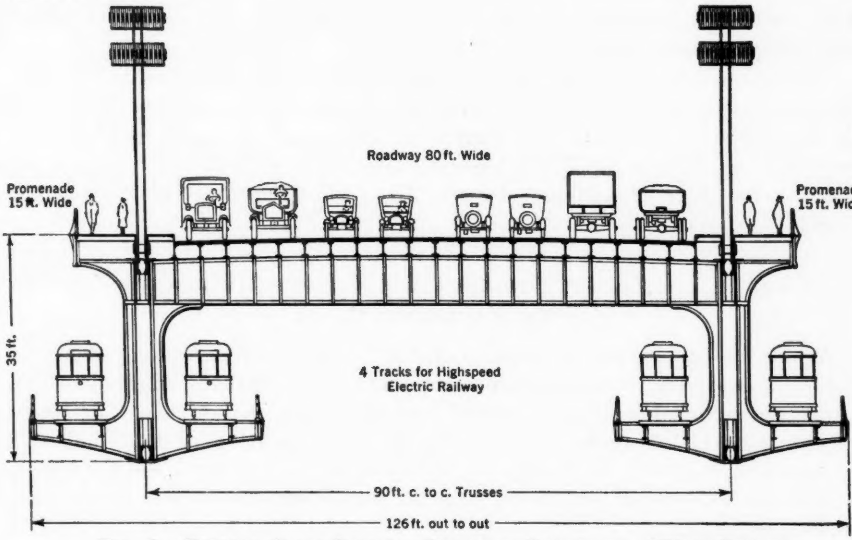


FIG. 6.—TYPICAL CROSS-SECTION, PROPOSED DESIGN FOR 179TH STREET SUSPENSION BRIDGE

The Acts carried an appropriation of $100 000 from each State to enable the Port Authority to undertake the preliminary surveys and studies. In the same year, Congress also passed an Act authorizing the Port Authority to build this bridge, subject to the approval of the War Department.

Work on preliminary studies was started in July 1925. A tentative report on the physical and financial feasibility of the undertaking was sent to the Governors of the two States in February, 1926, and thereupon the two States enacted further legislation pledging to the Port Authority, in aid of the prompt and economical construction of the bridge, the sum of $5 000 000 from each State. The Legislatures also voted an additional appropriation of $50 000 each to permit the completion of the preliminary work.

The statutes provide that moneys advanced by the States, together with interest at the rate of 4%, are to be repaid eventually by the Port Authority from the revenues and tolls arising out of the use of the bridge. The remainder of the funds required for the construction is to be raised by the Port Authority on its own securities.

[9] Port Authority Statutes.

Having early in the same year established the practicability and success of this method of financing in connection with the two interstate bridges across the Arthur Kill, the Port Authority in December, 1926, authorized for the Hudson River Bridge an issue of $60 000 000, Port of New York Authority, New York and New Jersey Interstate Bridge Gold Bonds, $20 000 000 of which were sold to a group of underwriters headed by the National City Company of New York. The 4% bonds were sold to the public on the basis of an interest yield of 4.2 per cent. They are secured by a first lien in the revenues which will be derived from the tolls and are to be amortized out of a sinking fund fıom the revenues within 25 to 30 years. An additional issue of 4½% bonds was sold by the Port Authority in October, 1929, when construction was well under way.

The plans for the George Washington Bridge were submitted to the War Department in December, 1926, and, after a public hearing conducted by Col. R. R. Ralston, U. S. District Engineer, they were promptly approved by the Chief of Engineers, Major General Edgar Jadwin, M. Am. Soc. C. E., and for the War Department by the Hon. Hanford MacNider, Assistant Secretary of War.

Geographical and Traffic Situation, and Economic Justification of the George Washington Bridge

While the State Acts specified the general location of the bridge, the Port Authority considered it essential to determine by careful investigation whether a crossing in that locality was needed and economically justified. The State Acts, moreover, left entire freedom in the determination of the kind and volume of traffic which the bridge was to accommodate. These questions could be answered only on the basis of a thorough survey of existing and prospective traffic conditions.

In reviewing the economic and traffic situation it must be kept in mind, in view of earlier conclusions reached relative to the most advantageous location of a bridge, that, within the past few decades, large centers of population have grown up in the northern part of Manhattan, the Borough of the Bronx, and Westchester County, while on the opposite side of the Hudson River there still remain comparatively undeveloped areas which strongly attract an overflow of population from the congested centers. The mere prospect of the coming of the bridge stimulated tremendous activities in the development of those areas years in advance of its completion. There are in Northern New Jersey also important industrial centers (Fig. 7), as Paterson, Passaic, Hackensack, etc., which have developed a rapidly growing volume of commercial traffic to and from New York City.

If the developments may be taken as an indication of social and economic demands of the population, and if the resulting appreciation in value of real estate alone in the territory contiguous to the bridge, even to date (1932), may be taken as a measure of economic benefit to the people, then the bridge has already more than paid for itself and has demonstrated, in the broadest sense, the economic justification of its construction. Not only that, but this

FIG. 7.—PRINCIPAL CROSSINGS AND ARTERIAL HIGHWAYS IN THE PORT OF AUTHORITY DISTRICT.

rapid accession in values, although largely stimulated by the wave of prosperity (1922–1929) plainly indicates the need and justification for additional crossings within the Metropolitan District.

Of course, the Port Authority could not depend on speculative or even assured increase in property values or other benefits to the people; its justification for the construction and investment of the capital had to be based solely upon the prospective revenue from reasonable toll charges.

In passing upon the justification of the location of a crossing so far up town, it must also be considered that the most urgent need for more adequate means of crossing the river in the down-town section of Manhattan was at that time in prospect of being met there by the impending completion in 1927 of the Holland Tunnel.

The traffic studies of the Port Authority and others revealed indeed the urgent necessity and justification of a crossing between Upper Manhattan and Northern New Jersey to take care of the rapidly growing vehicular traffic across the river in that vicinity and to relieve the situation growing out of the inadequacy of ferry transportation. The anticipated traffic developments have since been realized beyond expectation, as will be seen from the recorded increase in volume of trans-Hudson traffic.

Besides meeting local traffic demands, it was recognized that the bridge would form an important link in the arterial highway system in the two States and beyond. While the bridge accommodates principally traffic between Northern New Jersey and New York State west of the Hudson and New York City, it also serves traffic from Southern New Jersey, the Atlantic seaboard, and Eastern Pennsylvania, to points in New York State east of the river, New England, and Canada. This long-distance traffic thus avoids passing through the most congested parts of New York City.

In conjunction with the Washington Bridge across the Harlem River and probable future crossings over that river, and the Tri-Borough Bridge across the East River, the George Washington Bridge establishes an uninterrupted highway artery between Northern New Jersey and Long Island.

The traffic studies indicated clearly that the revenue from vehicular traffic alone would be sufficient to cover operating charges, interest, and amortization, of a bridge designed for a capacity to accommodate vehicular as well as rail-passenger traffic far in excess of that required for many years after the opening of the bridge to traffic.

Estimates of traffic across the Hudson River were made by the Port Authority Staff for each year from 1924 to 1960, a period of thirty-seven years. These estimates were based upon actual counts of the number and type of vehicles and their origin and destination during the average and peak months of traffic on the seventeen Hudson River ferries between the Battery, New York City, and Tarrytown, N. Y., in 1925.

The Holland Tunnel was opened to traffic on November 13, 1927. Its probable effect had to be taken into consideration in estimating the traffic across the George Washington Bridge. On the other hand, due allowance was justified for traffic that would be generated by the development of the territory contiguous to the bridge and of Bergen County generally. Careful

records were kept of the Hudson River traffic in subsequent years, and these were supplemented by additional counts in 1929, and a complete review of the distribution of the Hudson River traffic in 1930 in connection with studies for the proposed Midtown Hudson Tunnel in the vicinity of 38th Street, Manhattan.

Table 1 gives, for a number of years, the volume of vehicular traffic across the Hudson River from the Battery, in New York City, to Tarrytown, and the volume that would be diverted to the bridge, as forecast in 1926 at the time when the first bond issue was sold.

In comparison, the recorded actual volume of trans-Hudson vehicular traffic is shown for a number of years from 1920 to 1930. It is significant that, despite the general economic depression which set in in 1929 and continued through 1930, this traffic increased about 25% within those two years and nearly 100% within the five years from 1925 to 1930, or at a rate almost twice as fast as that forecast in 1926. The increase was most rapid at the crossings at and north of 42d Street, the region which the George Washington Bridge will serve most directly. Furthermore, the increase in population and motor-vehicle registration has been particularly rapid in the territories lying nearest the bridge. Population increased 73% in The Bronx, 82% in Westchester County, and 82% in Bergen County, during the ten-year period from 1920 to 1930, as compared to the 29% total increase for the Metropolitan District. In the 5-year period between 1924 and 1929, motor-vehicle registrations for

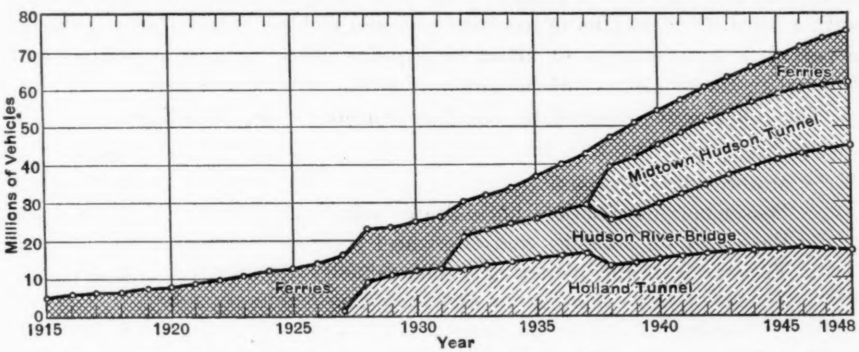


FIG. 8.—RECORDED AND ESTIMATED ANNUAL VEHICULAR TRAFFIC FOR ALL HUDSON RIVER CROSSINGS.

The Bronx, Westchester County, and Bergen County increased 95%, 87%, and 172%, respectively, as compared to 77% for the Metropolitan District as a whole.

In view of these developments—and even taking into account the effect which the proposed Midtown Hudson Tunnel in the vicinity of 38th Street, Manhattan, would have upon the flow of traffic over the George Washington Bridge (see Fig. 8)—it may be confidently expected that the traffic across the latter will exceed the figures forecast in 1926. This is reflected in the 1930 forecast of traffic, across the Hudson River and across the bridge as shown in the last two columns of Table 1.

TABLE 1—RECORDED AND ESTIMATED VEHICULAR TRAFFIC ACROSS HUDSON RIVER FROM THE BATTERY, NEW YORK CITY, TO TARRYTOWN, N. Y., AND ESTIMATED TRAFFIC ACROSS GEORGE WASHINGTON BRIDGE.

Year	1926 forecast of number of vehicles across Hudson River	1926 forecast of number of vehicles to cross on George Washington Bridge	Recorded number of vehicles across Hudson River to 1930	1930 forecast of vehicles across Hudson River, considering effect of opening of Midtown Hudson Tunnel, in 1937	1930 forecast of number of vehicles to cross on George Washington Bridge considering effect of Midtown Hudson Tunnel
1920......	....	....	7 660 000	....	....
1924......	....	....	11 890 000	....	....
1925......	....	....	12 570 000	....	....
1926......	14 200 000	....	13 740 000	....	....
1927......	15 500 000	....	16 090 000	....	....
1928......	16 900 000	....	20 720 000	....	....
1929......	18 300 000	....	23 550 000	....	....
1930......	19 800 000	....	25 500 000	....	....
1932......	22 700 000	8 600 000	....	30 100 000	8 700 000
1934......	25 600 000	9 800 000	....	34 200 000	10 100 000
1938......	30 600 000	12 000 000	....	47 000 000	11 500 000
1942......	34 500 000	13 500 000	....	60 400 000	18 600 000
1946......	37 400 000	15 000 000	....	71 000 000	25 100 000

Based upon the 1926 forecast of traffic and an average toll rate of 50 cents per vehicle (which is a fair approximation of the average rates charged on the ferries in the vicinity of the bridge), and 5 cents per passenger in vehicles, exclusive of driver, the annual net revenue was estimated to increase from a minimum of $5 250 000 in 1932, the first year of operation, and to be sufficient with a substantial surplus to meet interest and sinking-fund payments, during a period of amortization of about twenty-five years, as well as refund with interest to the two States of the amounts advanced by the latter.

The toll charges adopted by the Port Authority are, as follows:

Vehicle	Rate
Motorcycles; bicycles	$0.25
Passenger automobiles; horse-drawn vehicles (2 axles)...	0.50
Passenger automobiles and two-wheel trailers (3 axles)..	0.70
Trucks up to and including 2 tons capacity (2 axles)....	0.50
Trucks of more than 2 tons and including 5 tons capacity (2 axles)	0.75
Trucks of more than 5 tons capacity (2 axles)..........	1.00
Tractor and trailer; truck, 6 wheels (6 wheels and 3 axles)	1.25
Tractor and trailer; truck and trailer (8 wheels and 4 axles)	1.50
Buses, 4 wheels	1.00
Buses, 6 wheels..	1.10
Pedestrians	0.10

Traffic Capacity of Bridge.—Based on the estimated traffic figures and a capacity per lane per peak hour of 1 400 vehicles on the bridge proper, it was estimated that the vehicular traffic for the first five years could be accommodated conveniently on a four-lane roadway. Thereafter, it probably will

be necessary to increase the capacity. While it is not expected that the bridge would eventually carry a traffic in excess of 20 000 000 vehicles, it was considered advisable to provide for ample margin by the doubling of the initial capacity, which would be sufficient to accommodate at least 25 000 000 vehicles annually. Moreover, the additional roadway lanes will make it possible to segregate slow and fast moving traffic and thus permit greater speed, safety, and convenience of travel.

While the predominant necessity of the bridge for vehicular traffic was recognized, the possibility that it might be of service to rail passenger traffic was given careful consideration. With present-day tendencies to transport people in automobles and buses, that mode can be relied upon to take care of whatever demand there will be for passenger transportation across this bridge for a number of years. There can be little doubt, however, that with the growing up of a large population contiguous to the bridge the more efficient and economical transportation of people by electric rail service will become a necessity. Whether, and how, this traffic is to be carried over the bridge is yet a subject for study by the proper transit authorities in the two States.

The studies of the Port Authority's Staff indicated that the prospective volume of traffic fully warranted the comparatively small expenditure which was necessary to provide for four rapid transit tracks on the bridge, and such provision, therefore, was made in the design. It is believed that this provision is ample and all that may be reasonably justified at the present time (1932).

Topographical Conditions, Surveys, and Borings

A superficial examination of the topography at the site selected indicates the favorable conditions for a bridge situated within the limits defined in the Legislative Acts. The ground on both sides is high; that on the New Jersey side reaches, at the top of the Palisades, only about 500 ft. from the shore line, an elevation of about 280 ft. (Fig. 9), and that on the New York side rises, on the Washington Heights ridge, to a general elevation of about 200 ft., approximately 1 000 ft. from shore (Fig. 10). This permits of comparatively short, inexpensive approaches (Fig. 11); and, furthermore, the location in that vicinity does not involve extensive destruction of highly improved property as compared with location farther south.

An elevation of the upper roadway on the bridge, of about 240 ft., leaving 200 ft. clear height under the bridge, therefore, conformed with the general topography and incidentally provided ample clearance for all shipping that is likely ever to pass under it.

A glance at Fig. 24, introduced subsequently, indicates that obviously, on account of the narrowing of the river, the location of the bridge at the extreme point of Fort Washington Park is the most favorable one, requiring the least length of span. However, in order to determine conclusively, within the limits defined by the State Acts, not only the most favorable exact location of the bridge, but also the most feasible and economical arrangement of the main structure and the approaches, and reliable estimates of cost, it was

essential to undertake an accurate preliminary topographical survey of the vicinity, including a triangulation across the river, and borings to establish the subsurface conditions, more particularly the depth to solid rock. The surveys were embodied in a large map to the scale of 1 in. = 100 ft., and subsequently used also for the preparation of detailed maps to the scale of 1 in. = 20 ft.

Sixteen borings, all carried well into the solid bed-rock, were made late in 1925 at three tentatively selected locations, namely, in the vicinity of 181st Street, 179th Street, and 175th Street, Manhattan, respectively. At all three locations solid rock was found near the westerly pier-head line at depths ranging from 115 to 170 ft., with the surface of the rock falling sharply toward the river. In the borings made 500 ft. and more beyond the westerly pier-head line and carried to a depth of 200 ft., only silt was encountered. On the New York side the solid rock bed forms the shore, but its surface also falls off sharply toward the river.

These borings confirmed the assumption that, between the pier-head lines then established by the War Department, the bed-rock is too deep to permit of economical construction of bridge piers and that such piers must, and can, be placed between the pier-head lines and the shore, or on the shore. Preliminary and comparative plans and estimates were based on the results of these borings, and they confirmed the superior economy of the finally selected location on a line between 178th and 179th Streets.

As soon as the plans for the financing were effectuated, late in 1926, thirty additional borings were undertaken at the selected site of the New Jersey Tower. These borings were recorded on a glass model and revealed a surface of rock with a fairly uniform general slope of about 30° falling toward the river, but with contours practically parallel to the shore, and with a depth, within the area of the pier foundation, ranging from about 35 ft. at the southwest corner to a maximum of 75 ft. in the northeast corner.

While the results of the final borings indicated the feasibility of shifting the assumed pier location slightly toward the river to a depth to rock of about 100 ft., and thus shortening the span, this was not considered advisable, nor of any material benefit. On the contrary, the fact that the maximum depth to rock was only 75 ft. and the average depth less than 50 ft., made it appear feasible to build the foundation within an open coffer-dam. This method of foundation was adopted upon recommendation of Daniel E. Moran, M. Am. Soc. C. S., Consulting Engineer on Foundations, in competition with the pneumatic process which, to that time, had been considered the most practicable and the safest method. As a result of competitive bidding, the coffer-dam method proved to be about \$250 000 less expensive.

The borings also indicated, and subsequent exposure of the rock surface confirmed, a mostly hard and compact rock structure and an absence of open seams and large boulders which might have made the open coffer-dam method very difficult, if not impracticable.

Additional borings were made also at the sites of the tower and anchorage on the New York side, and they confirmed the assumption that both structures would rest on a solid bed of hard Hudson schist. An examination

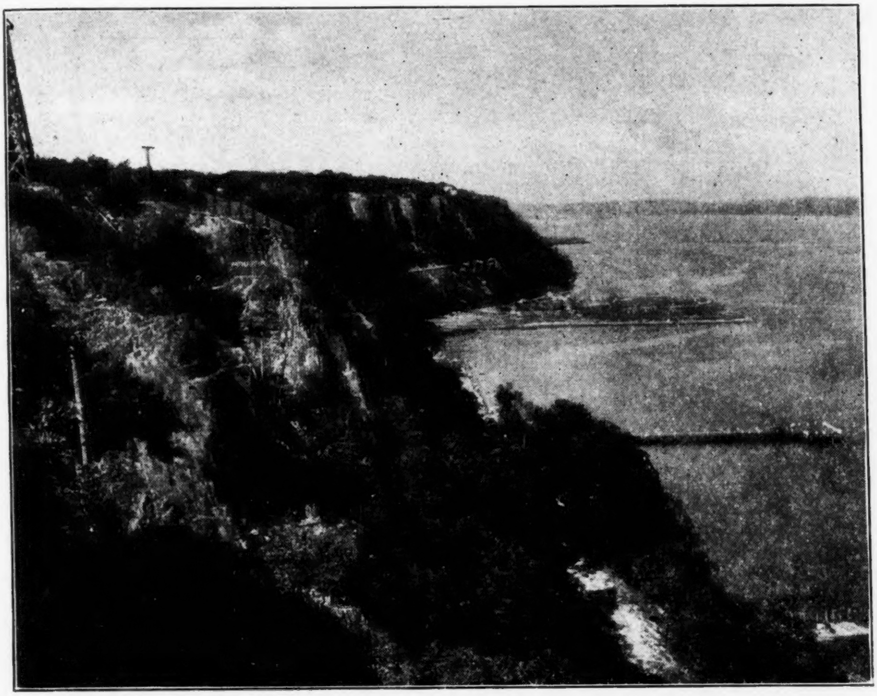


FIG. 9.—"PALISADES CLIFFS," 300 FEET HIGH, ON SITE OF BRIDGE, ON THE NEW JERSEY SIDE.

FIG. 10.—"WASHINGTON HEIGHTS," MORE THAN 200 FEET HIGH, NEAR SITE OF BRIDGE ON MANHATTAN SIDE.

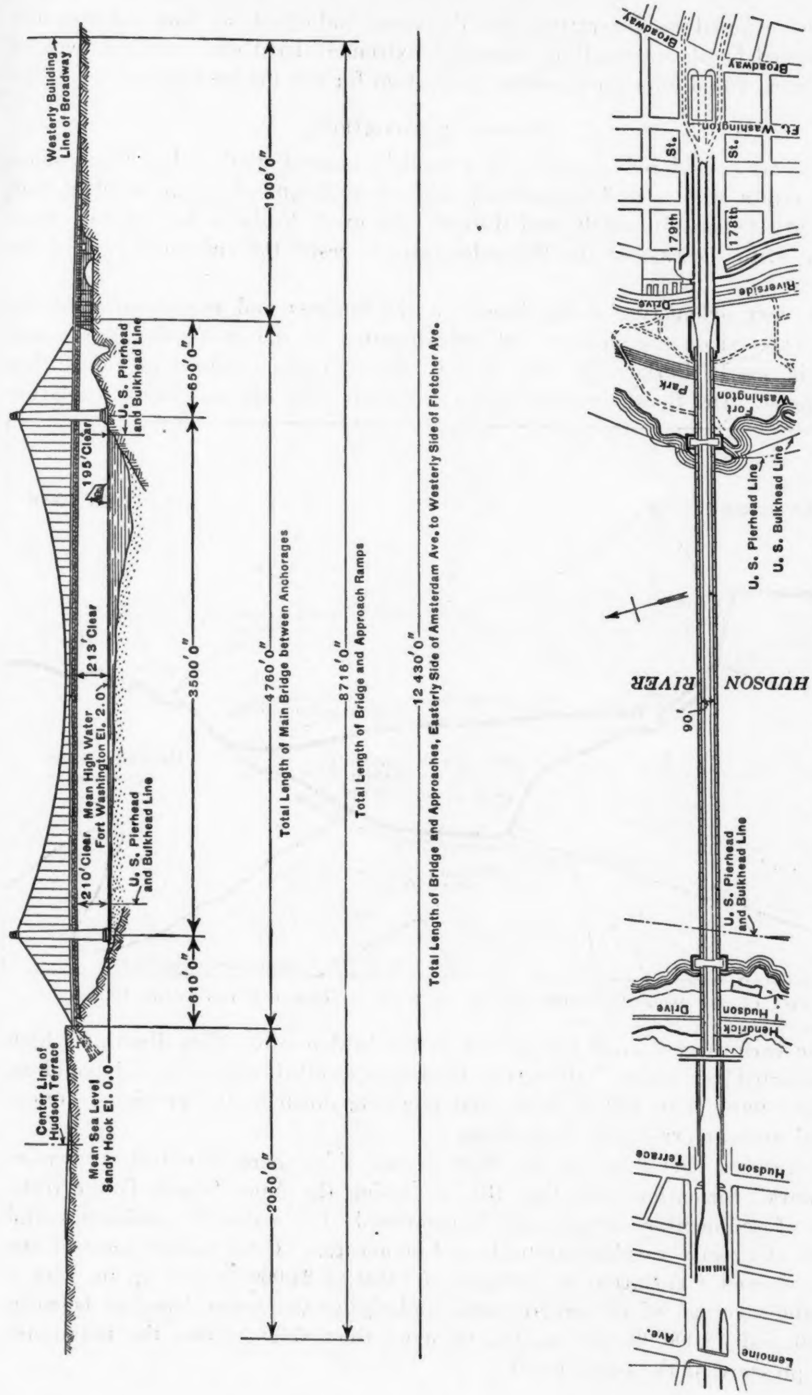


FIG. 11.—GENERAL PLAN AND ELEVATION, GEORGE WASHINGTON BRIDGE.

of the exposed rock forming the Palisades indicated, as was subsequently confirmed by the excavation, that that extremely hard and compact mass of trap-rock would offer an excellent anchorage for the bridge cables.

Geological Conditions

It was considered essential to establish, beyond doubt, that the various rock strata were sound geologically and of sufficient strength, solidity, and stability to sustain safely and durably the great loads to be imposed upon them or, in the case of the Palisades trap, to resist the enormous pull of the cables.

A very exhaustive study, based on the borings and examination of the rock exposed on the surface and, subsequently, in the excavation, was made by Charles P. Berkey, M. Am. Soc. C. E., who was retained as Consulting Geologist. Dr. Berkey constructed a diagram (Fig. 12) showing the relation

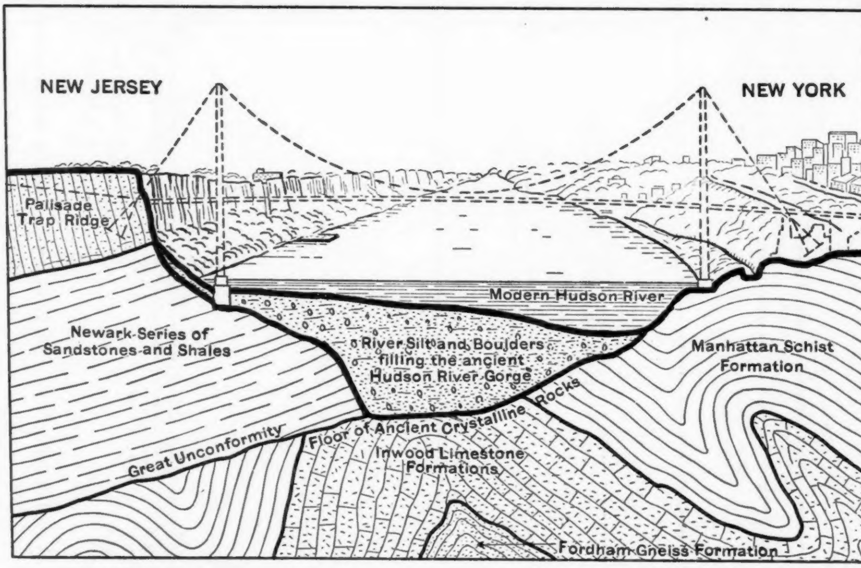


Fig. 12.—Geological Cross-Section at Site of George Washington Bridge.

of the various geological formations at the bridge site. This diagram which is distorted vertically, indicates a river gorge, filled with silt and boulders, perhaps more than 400 ft. deep, and reaching down to the previously unexplored ancient crystalline rock floor.

Overlying this floor on the New Jersey side, there is a bed, known as "Newark" formation (see Fig. 13), on which the New Jersey Tower rests. It is of sedimentary origin and is composed of a series of sandstones and shales of greatly varying strength and structure. (Compression tests of the cores showed a variation in strength of 3 000 to 24 000 lb. per sq. in. with a probable average of the entire mass underlying the tower base, of between 12 000 and 15 000 lb. per sq. in., or more than thirty times the maximum edge pressure of the tower base).

The borings indicated that the beds are slightly tilted, by an average of 10 to 15° from their original horizontal position, and that they dip toward the west beneath the Palisades. This is a very favorable condition in so far as the tower is concerned, in that it precludes any tendency of a portion of the bed-rock to slide toward the river gorge, such as might exist in case the beds would dip with a steep inclination toward the gorge.

This condition was confirmed by subsequent examination of the actual rock surface after it was exposed by excavation of the overlying loose material within the coffer-dam. The exposure of the rock surface also revealed (as was to be expected), a very jagged surface with sharp steps falling off toward the east. This condition undoubtedly has been caused by the erosion of the softer beds between the harder ones. It also has proved to be favorable for the tower foundation in that it obviated the cutting of artificial steps in the rock, which otherwise might have been necessary in order to avoid any tendency of the tower to slide on the rock surface toward the river. In fact, the surface of the solid rock, after removal of all loose and disintegrated portions, secured such an excellent bond with the concrete base, that it was deemed unnecessary to remove any part of the solid rock.

The exposed rock upon which the tower base rests, proved to be predominantly of the hard, coarse, sandstone type which has more than ample margin of strength to carry the load.

The Palisades trap, in which the New Jersey anchorage is embedded, overlies the Newark formation. As forecast by the geologist it proved—in the excavation for the 40-ft. approach cut and for the anchorage tunnels reaching down 250 ft.—to be a compact mass of hard diabase, an igneous rock of volcanic origin (Fig. 14). Only near the surface, as a result of long exposure, since the Glacial Age, is it found to be seamy and disintegrated.

All the ledges exposed on the New York shore are of the type known as Manhattan schist, a crystalline micaceous rock forming most of Manhattan Island. At the site of both the tower (Fig. 15), and the anchorage (Fig. 16), this rock was found by borings and subsequent excavation to be eminently sound and free of zones of local weakness which have given considerable trouble elsewhere.

Type of Bridge and Span Arrangement of Main Structure

Topographical and geological conditions indicating so clearly the points of support, it was a comparatively simple task to determine the type and general span arrangement of the main structure.

It was natural, in order to reduce the river span to an economical minimum, to place a pier on the extreme rocky point of Fort Washington Park which is also close to the United States pier-head line and from which the bare rock surface falls off steeply. For reasons already mentioned, a pier on the New Jersey side had to be located about 200 ft. back of the pier-head line, thus fixing the river span at 3 500 ft.

This great span has given rise to criticism that it was extravagant, and that a more economical structure was feasible and permissible by moving the

New Jersey pier about 800 ft. out into the river, which is fairly shallow for a considerable distance from the shore, although the rock surface falls off to a depth of more than 200 ft. at that point.

Without regard to the possible adverse effect on shipping by a pier in the river, and without allowance for the hazards and uncertain cost and time of construction involved in building a suitable pier foundation to such great depth, a conservative analysis of cost should convince any one that the adopted span arrangement is the most economical under the given conditions.

In an endeavor to determine the effect which the shifting of the New Jersey pier would have on the cost of the entire structure, the writer discovered that for the slope of the rock surface as found, the saving which could be effected in the superstructure by decreasing the central span was practically offset by the increased cost of the deeper foundation. This comparison would not be so favorable, however, to the greater span if the conception of a rigid stiffening system formed the basis of proportioning, because under that theory the cost of the superstructure increases at a much faster rate.

There appears to be a widespread, but unwarranted, impression in the minds of engineers and others that length of span is the predominant factor in the economy of a large bridge; whereas, in many cases, such as that of the George Washington Bridge, a careful and rational analysis of conditions and costs would indicate that length of central span is a lesser factor.

It was also quite obvious that the face of the solid Palisades cliffs, about 650 ft. westerly of the New Jersey pier, fixed the location of an abutment or anchorage of a suspension bridge. The high rocky ground in Fort Washington Park, on the New York side, offered a symmetrically located, but not quite as favorable, point for an anchorage on that side. This location of a hugh abutment brought forward well-intentioned misgivings on the part of those interested in the preservation of public parks, and a demand was made that the abutment be moved to the still higher ground easterly of Riverside Drive.

The demonstration that the use of Fort Washington Park would not be materially impaired, that a more easterly location of the anchorage would result in a much longer side span, with consequent dissymmetry of the main structure, and deep, conspicuous, stiffening trusses over the Park and Riverside Drive (which would be much more objectionable from the æsthetic point of view than a well-designed massive anchorage and arch structure over the Drive, and would add millions to the cost), finally appeased the opponents and secured the approval of the City Government to the proposed arrangement.

The great central span and the possibility of the construction of solid, comparatively inexpensive, cable anchorages should force any student of the economics of long-span bridges to the conclusion that a rationally designed suspension bridge would be economically superior to any other conceivable type, not considering its superior æsthetic merits in that particular landscape.

This might not have been so always, and it is quite evident that even to-day engineers have different conceptions as to the relative economic merits of different types for long spans. Indeed, if the many designs are compared,

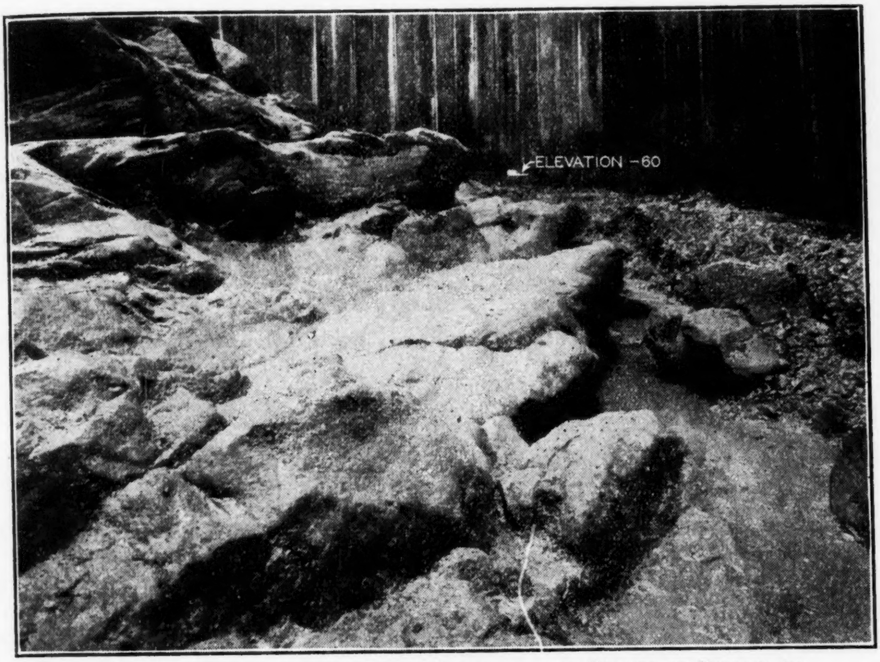


FIG. 13.—NEWARK FORMATIONS OF SANDSTONE AND SHALES AT BOTTOM OF NEW JERSEY TOWER.

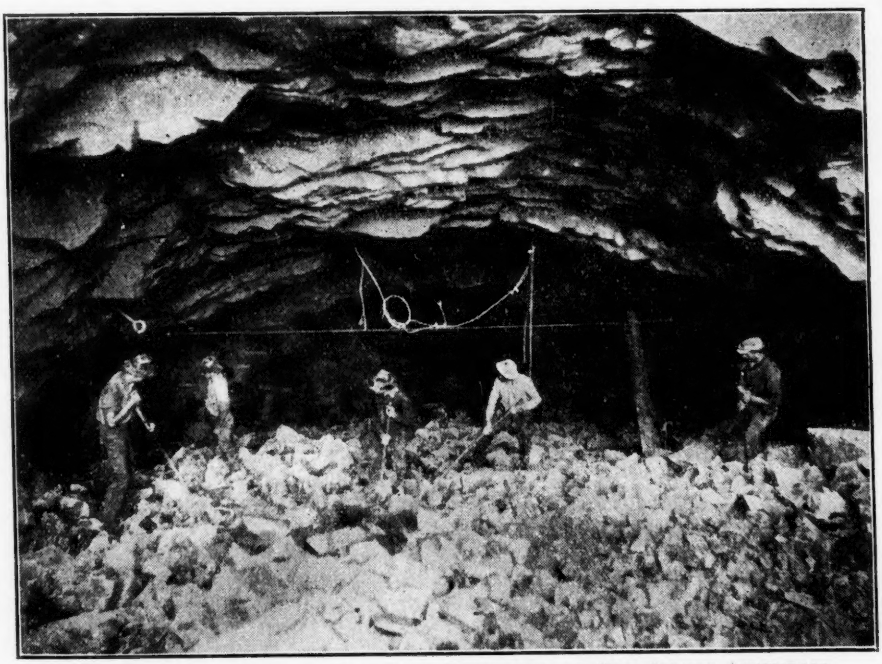


FIG. 14.—SOLID ROCK, OR TRAP-ROCK FORMATION, AT NEW JERSEY ANCHORAGE.

FIG. 15.—MANHATTAN SCHIST AT BOTTOM OF NEW YORK TOWER, NEAR SURFACE.

FIG. 16.—BED OF MANHATTAN SCHIST AT SITE OF NEW YORK ANCHORAGE.

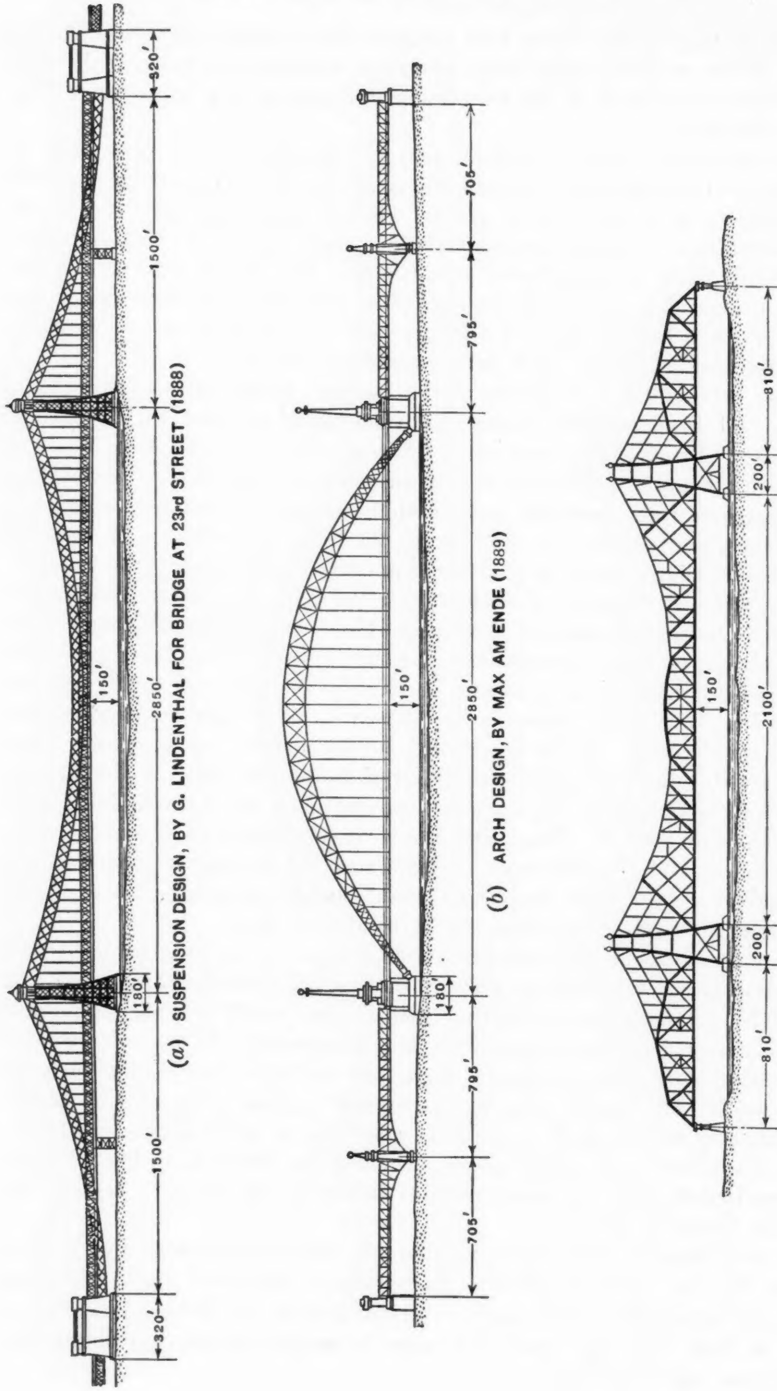
which from time to time have been proposed for a bridge across the Hudson at New York, or across other wide navigable streams, a perplexing diversity of conceptions is found of the best design for such a long span. (See Figs. 17, 18, and 19.)

The cantilever, pure or hybrid, may be dismissed as a possibility by referring to the exhaustive investigations of the War Department in 1894, as referred to elsewhere in this paper. The structure then proposed by the New York and New Jersey Interstate Bridge Company was to be a cantilever (Fig. 17(*c*)) with a central span of 2 000 ft. Comparing this with a suspension bridge of a type shown in Fig. 18(*a*), the Board of Engineers found that even for a central span of 3 200 ft. the suspension type would not be materially more expensive than the proposed cantilever.

Under present-day conceptions of the rational design of the two types of bridges, and in particular under conditions such as exist at the George Washington Bridge, the economic difference between the two types is materially accentuated in favor of the suspension type. The superiority of the suspension type over the arch, both economically and æsthetically, is less obvious, and the site of the George Washington Bridge, with its solid rocky shores, might invite an investigation of the latter type. The comparison of an arch with a span of 1 685 ft. over the Kill van Kull in 1928 proved this type to be more economical than that which was considered an equivalent suspension bridge with a central span of 1 522 ft., largely because of expensive anchorages required by the latter on account of the low level of the ground. At Fort Lee, both arch abutments would have to be set several hundred feet back of the location of the suspension bridge towers—on the New York side to satisfy clearance requirements for shipping, on the New Jersey side to find solid rock nearer the surface—and the span of an arch would thus become close to 4 000 ft. This, with the relatively inexpensive cable anchorages on account of the high rocky ground and the increasing economy of a suspended structure with increasing span, should leave no doubt of the economic superiority of the suspension type at Fort Lee.

A remarkably bold and very creditable design for an arch bridge across the Hudson River was made in 1889 by the English engineer, Max am Ende, (Fig. 17(*b*)). He claimed at the time that his arch would be more economical than the suspension type proposed by Mr. Lindenthal. In accordance with present-day conceptions in design there can be little doubt that such an arch would be very much more expensive, and a great mass of steel at the height of about 600 ft. above water level would not be as attractive in appearance as a graceful suspension bridge, provided the latter is without clumsy stiffening trusses, such as were embodied in several of the early designs for a Hudson River bridge.

All later designs show preference for the suspension type. For a span of 3 500 ft., and under conditions permitting a relatively light stiffening system and inexpensive anchorages, as in the case of the George Washington Bridge at Fort Lee, the suspension type is unquestionably far more economical than any other type.

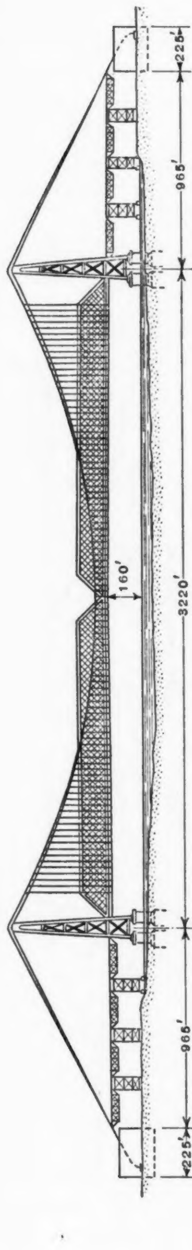


(*a*) SUSPENSION DESIGN, BY G. LINDENTHAL FOR BRIDGE AT 23rd STREET (1888)

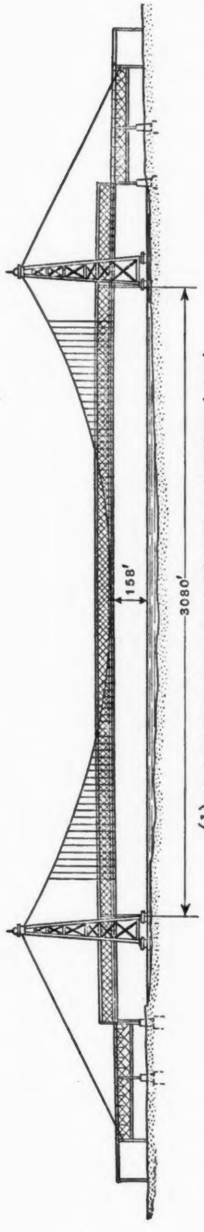
(*b*) ARCH DESIGN, BY MAX AM ENDE (1889)

(*c*) CANTILEVER DESIGN, BY UNION BRIDGE CO. FOR BRIDGE AT 70th STREET (1893)

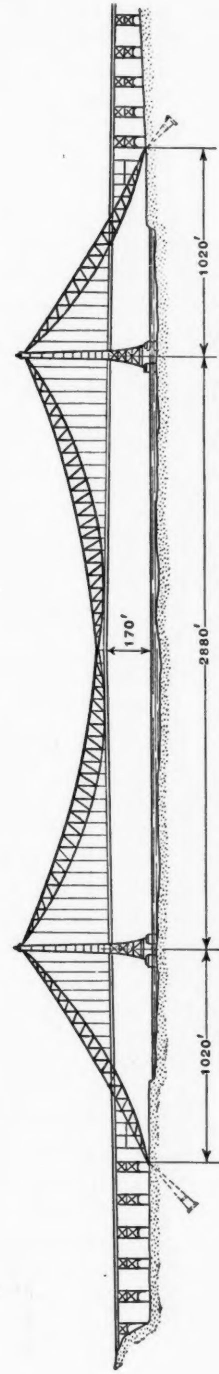
FIG. 17.—EARLY STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.



(*a*) SUSPENSION DESIGN BY THE BOARD OF ENGINEERS APPOINTED BY THE SECRETARY OF WAR (1894)



(*b*) SUSPENSION DESIGN BY GEORGE S. MORISON (1896)



(*c*) SUSPENSION DESIGN BY BOLLER, HODGE AND BAIRD FOR BRIDGE AT 57th STREET (1913)

FIG. 18.—EARLY STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.

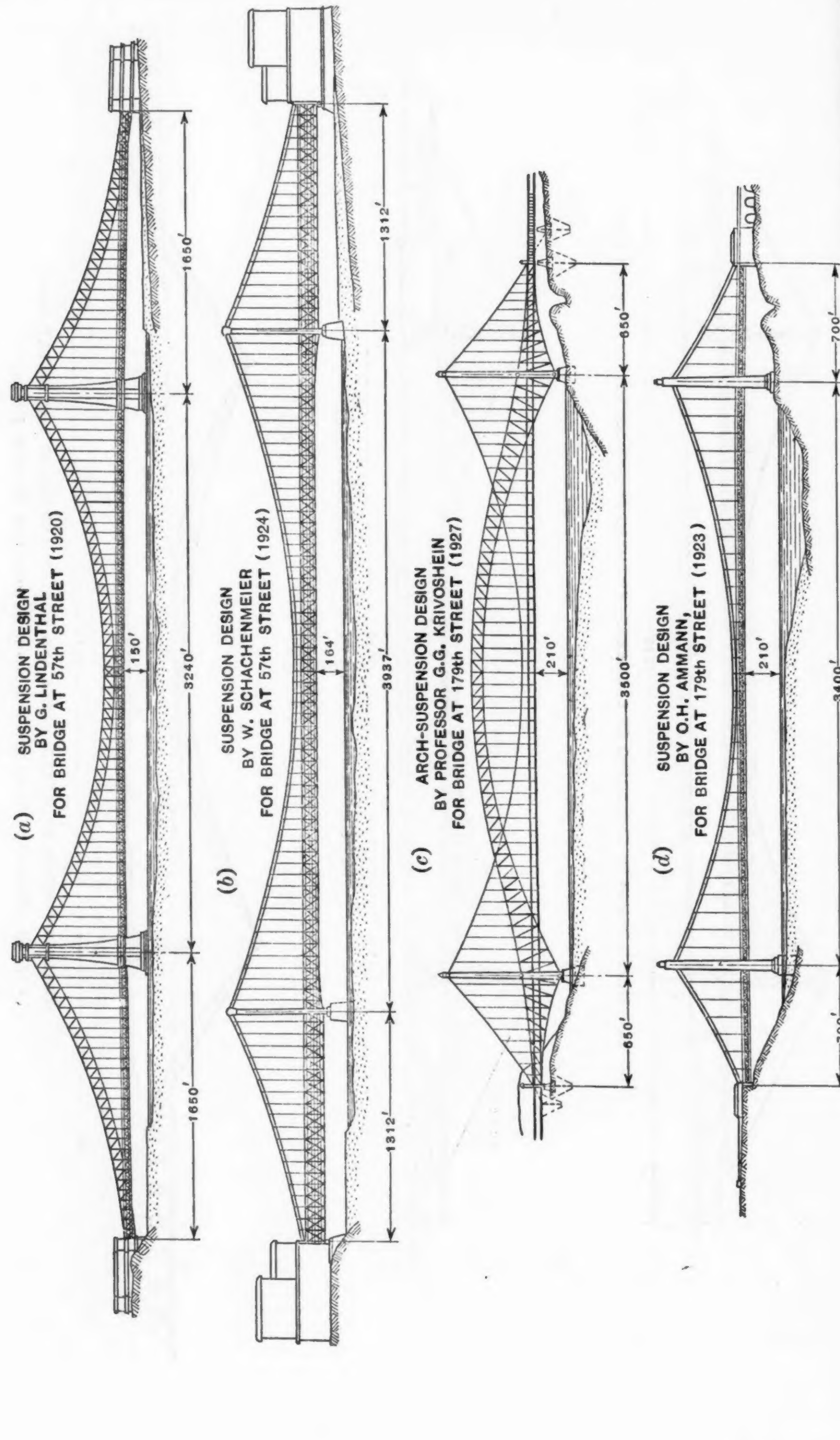


FIG. 19.—RECENT STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.

From time to time there have been proposed and likewise found application combinations of the pure suspension type and other types. Whatever may be the claims of scientific or economic merits of such hybrid types (and it is very doubtful in the writer's mind that they are justified because of the lack of structural simplicity and clearness of function of such incongruent systems), they cannot satisfy æsthetic principles.

Fortunately, while such types have found their way into the field of moderate spans where they form interesting structural creations, they have not advanced beyond proposals, in the field of long spans. Even Mr. John A. Roebling, troubled by the problem of finding means to provide sufficient rigidity, seriously considered a combination of a rigid truss with suspended cables, but his good common sense finally led him to the simple suspension design so admirably illustrated by the Brooklyn Bridge.

FIG. 19.—RECENT STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.

Fig. 19(*a*) shows an interesting design of a combination arch and suspension type, proposed in 1927 by Professor Gregory G. Krivoschein, of Prague, Czecho-Slovakia, to fit the location of the George Washington Bridge with a span of 3 500 ft. A combination of rigid cantilever truss with suspended cables and span of 3 900 ft. is illustrated in a study made in 1924 by the late Professor W. Schachenmeier, of Munich, Germany, to fit the location of a bridge across the Hudson River near 57th Street, Manhattan. (Fig. 19(*b*)).

Maximum Feasible Spans.—The unprecedented length of span of 3 500 ft. being exactly twice the longest suspension span in existence has given rise to the question on the part of laymen as to whether the building of this bridge is feasible.

Engineers familiar with the design and construction of large bridges have pointed out from time to time that the feasibility of building a bridge of a span as long as 3 500 ft. and more, is essentially a question of economy, and that the span length and size of a bridge has nothing whatever to do with its safety, either during erection or after completion.

The feasible limit of span is reached when the metal required to carry a given load becomes excessive in cost and not because the safety is impaired. The physical limit of span is reached when no amount of metal can safely carry more than its own weight. The latter limit can be mathematically determined for the safe strength of any given material, and load conditions. In the aforementioned investigation in 1894 by the Board of Engineer Officers, 4 335 ft. was found, upon conservative assumptions, to be "the maximum span practicable from the engineering point of view."

In acordance with present-day accepted views regarding the proportioning of stiffening trusses in long spans, the formula used by that Board results in greatly excessive weights of these trusses, and of the cables and towers which have to carry them. Furthermore, the permissible stress in the wire cables was assumed at only 60 000 lb. per sq. in., whereas with material of 240 000-lb. strength available to-day, and for very long spans, a permissible stress of 90 000 or 100 000 lb. per sq. in. would be entirely safe. In the light of these facts it may be demonstrated easily that a modern bridge of 10 000 ft. span could be built with perfect safety. It is only above this limit that the

cables increase rapidly in weight and cease to be practicable, if they are to carry any appreciable load in addition to their own weight. The practicable length of span for a cable to carry itself is, of course, much greater.

Conception of Type of Suspension System.—Thus, while the type of bridge and its span arrangement logically forced themselves upon the designer, he was yet confronted with the more complex and controversial questions of selecting the appropriate form and proportions of the suspension system. The great number of different systems which have been proposed and applied, indicate a wide diversity of conceptions and practices in respect to these questions. Figs. 17, 18, and 19 show only a few systems, those variously proposed for a bridge across the Hudson River, at New York City.

In the case of the George Washington Bridge, the controlling criteria in selecting the system and its proportions were structural simplicity, maximum economy consistent with the required degree of rigidity, competitive conditions, and æsthetic conception.

The first of these criteria led to the system which is unquestionably the simplest in its structural details, as well as for erection, namely, the plain cable with parallel chord stiffening trusses along the floor; but this system, whether the trusses be three-hinged as in the design of the Board of Engineers (Fig. 18(*a*)), or two-hinged as in most of the modern suspension bridges, or continuous through the towers, as in the design of Mr. George S. Morison (Fig. 18(*b*)), or cantilevers, as in the study of Professor W. Schachenmeier (Fig. 19(*b*)), is not economical in a long span, nor in conformity with the writer's conception of a graceful structure, if, as in some of the designs, the stiffening trusses are made very deep.

Furthermore, it is not necessary, in accordance with more recent views, to provide deep and rigid trusses in a long heavy span, and, particularly so, when the side spans are relatively short and the dead load relatively great, as in the case of the George Washington Bridge.

Extensive studies of the relative rigidity of similar structures and their behavior under actual conditions, and calculations of the degree of rigidity to be obtained in the selected system, led the writer to determine finally upon a very shallow and flexible truss, which not only resulted in far-reaching economy, but also effected a light and graceful appearance Fig. 19(*d*)). Incidentally, by keeping the top chords of the stiffening trusses in the plane of the upper or roadway floor, obstruction by these trusses of the splendid view of the landscape that will be had from the roadway was avoided.

Where a considerable degree of rigidity is required, the trussed system, as proposed by Mr. Lindenthal (Figs. 17(*a*) and 18(*a*)) and (to a lesser degree) the stiffened chain system as proposed by Messrs. Boller, Hodge, and Baird (Fig. 18(*c*)) are economical as far as material is concerned, but the erection of such suspended trusses unquestionably involves difficult and expensive erection operations and partly offsets the economy in material. Furthermore, in a large bridge, built by a public agency, it is essential that the widest possible competition be assured for all important parts of the structure, and the fact that the design for the George Washington Bridge lent itself equally

well to the use of wire cables and eye-bar chains was, therefore, an important factor in its adoption, and proved to be of decided advantage.

Finally, whatever influence these various considerations may have had on the general conception of the design, the writer has admittedly been influenced by his personal conceptions and taste. He has always been an admirer of the early English suspension bridges with their general simple appearance, their flat catenary, light, graceful, suspended structure, and their plain massive and, therefore, monumental towers.

Deviations from the simple unstiffened cables were due to the efforts to give the system greater rigidity. This has been accomplished by various more or less efficient expedients, such as inclined stays, connecting the floor directly with the top of the towers or other fixed points, by stiffening trusses placed along the floor or by stiffening systems attached to the cables themselves. In some cases the cable has even been combined with an upright rigid arch or with a cantilever truss.

It is significant, however, that after nearly a century of efforts to devise and introduce novel forms of suspension systems, or hybrids between the suspension type and other types, engineers have, in designing the longest modern suspension bridges, returned or adhered to the simple, naturally graceful forms which are characteristic of the early bridges of this type.

The Floor Structure, Stiffening Trusses, and Wind-Bracing

Consideration of the traffic requirements, the conception of the stiffening system, and the arrangement of the four cables in pairs on each side of the floor, led to a structurally and statically simple arrangement of the floor system suspended from the cables (Fig. 20).

An upper floor is designed to accommodate at least eight lanes of vehicular traffic. Beneath it, and connected to it by rigid floor frames, is a lower deck designed to carry at least four tracks for heavy rapid-transit traffic; or this deck may be utilized for additional vehicular lanes in case that should ever become necessary and desirable.

A shallow stiffening truss with chords only 29 ft. apart vertically is placed on each side of the floor system in the plane of the cables and suspenders. A single, relatively flexible, horizontal wind truss is arranged in the plane of the upper deck, the upper chords of the stiffening trusses forming the chords of this horizontal wind truss. The wind forces acting on the lower deck are transmitted to this wind truss through the rigid floor frames.

In a preliminary design the floor frame was conceived as an inverted **U**, with brackets cantilevering out from the vertical posts on both sides (Fig. 6). This arrangement was eventually abandoned in favor of the somewhat simpler and only slightly more expensive closed frame carrying all tracks inside the posts. This entire floor structure is designed so that the lower deck, together with the webs and bottom chords of the stiffening trusses, could be omitted initially and added in a very simple manner at any time in the future when necessity therefor will arise.

The vertical stiffening trusses have a depth of 29 ft. throughout, which is only one-one-hundred-twentieth of the central span. This compares with

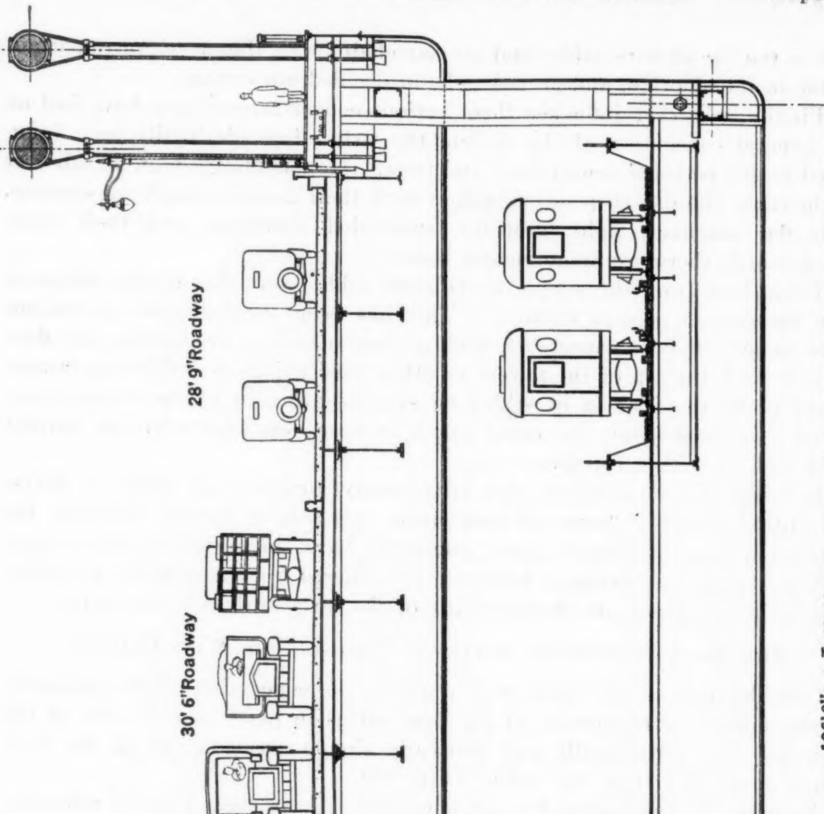


FIG. 20.—TYPICAL CROSS-SECTION OF DOUBLE-DECK FLOOR, GEORGE WASHINGTON BRIDGE.

the corresponding ratio of one-sixty-third in the Delaware River Bridge, which has a center span of 1 750 ft.; with one-sixtieth in the Manhattan Bridge, which has spans of 1 470 ft.; and with one-sixty-third of the central span of 1 630 ft. of the Bear Mountain Bridge, these being the three longest modern suspension spans in existence, or in course of construction, at the time the writer made the first studies. The first two, like the George Washington Bridge, are designed to carry highway and electric rail passenger traffic, while the Bear Mountain Bridge carries highway traffic only.

The Ambassador Bridge, in Detroit, Mich., built since that time, with a center span of 1 850 ft., has stiffening trusses 22 ft. deep, or one-eighty-fourth of the central span. The Golden Gate Bridge, in San Francisco, Calif., is designed with stiffening trusses 25 ft. deep, or one one-hundred-sixty-eighth of the central span of 4 200 ft. Both the latter bridges are designed for vehicular traffic only.

Stiffening trusses of the Manhattan and Delaware River Bridges may be called semi-flexible, while those of the George Washington Bridge are practically "flexible," that is, they exert almost no restraint upon the distortions of the unstiffened cables. On the contrary, they are forced to deform almost to the full extent and shape of the distortions of the unstiffened cables. The system in its initial stage, being without lower floor and stiffening trusses, is entirely flexible, that is, without any stiffening effect upon the cables.

The permissibility of an almost flexible system in the case of the completed bridge—that is, with rapid transit trains running over the bridge on the lower deck, or of an entirely flexible system in case the bridge carries only vehicular traffic on the upper deck—was not obvious to the writer at the inception of his studies.

The general tendency, both in the United States and abroad, had been toward a rigid stiffening system, and the textbooks and many modern treatises on suspension bridges had confined themselves entirely to that system and to the elastic theory, without respect to span length, dead weight of bridge, or character of traffic.

Extensive studies convinced the writer that for a long-span suspension bridge a rigid system was not necessary. He was also familiar with the fact that by the application of the correct or so-called deflection theory, as distinguished from the "elastic theory," to a more or less flexible system, material economies can be effected. This is inherently due to the stiffening effect of the dead load, which effect is ignored in the so-called elastic theory. The latter fact had been pointed out by various writers, notably, Professor Melan, of Vienna, Austria. It had also been proved by application of a modified deflection theory by Leon S. Moisseiff, M. Am. Soc. C. E., to the design of the Manhattan and Delaware River Suspension Bridges.[10]

Aware of the ample rigidity of the Manhattan Bridge under actual traffic conditions, and of the sufficiency even of the much more flexible Brooklyn Bridge under all ordinary conditions, the writer became convinced that in the George Washington Bridge—with its much longer and heavier center

[10] Final Rept. of the Board of Engrs. on the Delaware River Bridge, 1927.

FIG. 20.—TYPICAL CROSS-SECTION OF DOUBLE-DECK FLOOR, GEORGE WASHINGTON BRIDGE.

span (the latter weighing four times that of the Manhattan Bridge and ten times that of the Brooklyn Bridge), its comparatively shorter side spans with almost straight cables acting as rigid back-stays, and the more effective distribution of concentrated loads by reason of its wide rigid floor—stiffening trusses of relatively greater flexibility than those used in the aforementioned smaller bridges, were permissible and economically required.

As a result of lengthy theoretical investigations, supplemented by observations on mechanical models, made in an endeavor to find the appropriate degree of rigidity of the stiffening trusses for the George Washington Bridge, the writer came to the conclusion that the arrangement of nearly flexible trusses in the finished bridge, and the omission of trusses in the initial stage of a single highway deck, were perfectly permissible and would secure a degree of rigidity at least equivalent to that of any of the aforementioned large modern bridges.

In fact, the governing function of a stiffening system in a long span is to prevent excessive gradients of the floor due to deflection of the cable. For vehicular and electric passenger traffic the limiting grade, under severe loading conditions, can safely be assumed at 5%, in view of the improbability that it would ever be produced, and even then it would prevail only over very short stretches.

The deflection curves for the George Washington Bridge showed that, under a combination of extreme temperature change and live load, the grades in the perfectly articulated bridge would not exceed 2.4%, and in the stiffened bridge as designed would be less than 2¼ per cent. The function of the comparatively light flexible stiffening trusses was considered mainly to stiffen the floor locally.

In his studies the writer became aware of the fact that even the so-called exact or deflection theories (a number of which had been advanced by that time), became unreliable and inapplicable in the case of trusses of relatively great flexibility, and that the simplest and most reliable method was to calculate the deflections of the unstiffened cables and the corresponding bending moments produced in the trusses, and, subsequently, correct the results slightly to allow for the comparatively small stiffening effect of the trusses.

The economy effected by adopting flexible trusses is far-reaching, as may be seen by a comparison of the weight of stiffening trusses and wind-bracing compared to the weight of cables, the live load, and the total dead load per foot of bridge in the George Washington Bridge and three other bridges (see Table 2).

In the Manhattan Bridge the stiffening system amounts to 30% of the entire dead load and nearly 140% of the main carrying members, the cables and suspenders. In the completed George Washington Bridge the stiffening system amounts to less than 6% of the entire dead load and less than 20% of the weight of the cables and suspenders.

Some formulas would increase the weight of stiffening roughly in proportion to span and live load. On that basis, when compared with the stiffening of any of the bridges in Table 2, the system of the George Washington Bridge, with silicon steel chords, would weigh roughly from 13 000

to 14 000 lb. per ft. Actually, it is about one-sixth this comparative weight in the final condition and about one-twelfth in the initial condition. Considering also the fact that every dollar spent for steel in the floor and stiffening trusses in a span of this length requires at least an equivalent expenditure

TABLE 2.—COMPARISON OF DEAD LOAD WEIGHTS

Item	Manhattan Bridge	Bear Mountain Bridge	Delaware River Bridge	George Washington Bridge
Span length, in feet	1 470	1 630	1 750	3 500
Approximately equivalent live load, in pounds	11 000	2 500	7 000	8 000
Average weight of stiffening trusses and wind bracing, in pounds:				
For highway only	......	2 650	......	1 110
For highway and rapid transit	7 200	......	5 730	2 350
Grade of steel	Nickel, in chords	Carbon	Nickel, in chords	Silicon, in chords
Average weight of cables and suspenders, in pounds	5 300	1 500	4 650	12 530
Total average dead load, in pounds	24 000	11 500	23 700	40 000

for materials in the cables, towers, and anchorages to carry the floor steel, the total saving by the adoption of the flexible trusses is estimated to be almost $10 000 000.

This saving in the stiffening trusses of the George Washhington Bridge is not due entirely to their flexibility in a vertical plane, but somewhat to the comparative flexibility of the horizontal wind trusses the chords of which form the upper chords of the vertical stiffening trusses. Due to this lateral flexibility, and to the great weight of the cables, a large proportion of the wind load is transmitted from the wind truss to the cables and by the latter to the tops of the towers. Thus, the wind trusses become much lighter, and this economy, together with the corresponding saving of materials in the cables, towers, and anchorages, is only slightly offset by the extra material required in the towers on account of the increased lateral cable reaction at the top.

The studies indicated that under possible severest wind pressure acting over the entire length of the center span of 3 500 ft. (heaviest wind pressures have been observed to extend generally over a width not greater than 800 ft.), the maximum lateral deflection at the center, if the bridge were perfectly articulated and without lateral bracing, would not exceed 12 ft., or about one-three-hundredth of the span length, which would be entirely permissible. Actually, the rigidity of the floor and the inertia of the enormous dead weight resisting sudden gusts of wind prevent even such deflections, and the bridge as a whole would be perfectly safe and sufficiently rigid even without a wind truss. The comparatively flexible wind truss was provided mainly to prevent possible excessive local distortion of the floor and consequent high stresses in the floor members and their connections as a result of extraordinary wind effects.

Although no measurements of deflections have been made to date (1932), observations even on the partly completed bridge indicate plainly its remarkable lateral rigidity.

A fuller description of the stiffening system will be contained in another paper of this series. Reference is made also to a more complete discussion by the writer of the nature of the stiffening system of suspension bridges and its effect upon the economy of such bridges in his discussion[11] of the paper by J. A. L. Waddell, M. Am. Soc. C. E., entitled "Quantities of Materials and Costs per Square Foot of Floor for Highway and Electric Railway Long-Span Suspension Bridges."

The Cables and Their Anchorages

The conception of the type and arrangement of the cables is intimately tied up with that of the floor structure. As mentioned, it was desired to suspend the latter only in two vertical planes to secure greatest structural simplicity and determinateness of stress action. Owing to the consequent large concentration of load in each of the two planes of suspension, it was evident that, instead of a single large cable of a size far beyond any previously constructed, it was advisable to arrange a group of two or more cables. Such division into units was considered called for, also, because it was held possible that part of the group of cables would be erected first, and the remainder later, when increase in traffic capacity became necessary and justified the additional expenditure. This idea was finally abandoned in favor of the initial completion of the cables for full load capacity, although it involved an additional initial expenditure of about $8 000 000.

Such an arrangement raises the question of equal distribution of load between the various units of the group, whether they are separate wire cables or units of eye-bar chains. Not only laymen, but engineers have been puzzled over this question, and complicated structural devices have been proposed, and actually used, in order to secure uniformity of distribution of load and stress; yet this is one of the simplest problems and one which solves itself naturally where the cables or their units are free to deflect, because if the suspension of the floor is arranged so that the cable units must deflect equally, their stresses from vertical load must be the same. The uniformity of distribution is the greater the longer the span and the flatter the catenary.

Inequality of stress is caused mainly by inequality of temperature changes in different parts or units of a cable. Such temperature variations cannot be avoided, either in a single cable or a group of cables, and must be taken care of in the marginal strength; but it is the writer's belief that their effect has been largely exaggerated. In the case of the George Washington Bridge, various schemes of grouping the cable units and their sequence of construction were investigated and in part left for selection by the contractor. The arrangement finally determined upon provides for a pair of cables, 36 in. in diameter, on each side of the floor, the individual cables lying side by side, 9 ft. apart on centers. The type of suspension system selected lent itself to two radically different types of cables, the so-called wire cable and the so-called eye-bar cable or chain. In fact, this adaptability of the general design of the bridge to both these types was an important factor in its conception.

[11] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 932.

In a structure of this magnitude, especially when undertaken by a public body, and under conditions which prevailed in this particular case, the widest possible competition is essential and it was for this reason that upon recommendation of the writer, concurred in by the Consulting Engineers and the General Manager, the Commissioners of the Port Authority approved the preparation of competitive designs and the calling for competitive bids, not only for the two different types of cables, but for various alternate arrangements, such as cables of each pair placed side by side or one above the other; also, for cables erected simultaneously or successively, thus giving the bidders wide latitude in applying their ingenuity, experience, and facilities in developing the most economical methods of fabrication and construction.

The outcome of the bidding, which was in favor of the wire cable type as the lowest in cost to the Port Authority, indicated clearly that this course fully justified the additional time and expenditure involved in the preparation of competitive designs and was to the best interest of the Port Authority and that of the public. Such procedure, however, might not always and in all cases be justified, and the relative prices established by this competition may be taken as a fair basis for comparative studies for other projects at the present time (1932).

The question of the relative merits of the two types of cable has been the subject of considerable discussion and controversy among engineers from time to time, in the United States and abroad. It is a complex question and one which lends itself legitimately to personal preference, whether that be based upon impartial judgment, prejudice, or interest.

In so far as engineers are concerned, their impartial judgment (considering present knowledge based upon past performance of the two types), must take into consideration such important factors as the peculiarities of the case, the relative quality of material permissible, unit stresses, and other details of design. General condemnation of one or the other type on the vague grounds of greater or lesser weight, inferior safety, durability, or assumed greater cost, is open to justified criticism of partiality or prejudice.

In so far as the public is concerned, there are the outstanding facts that both types belong to the oldest forms of bridge members. Both have been developed in American bridge practice during more than a century; both have reached a high degree of perfection in material and structural details; and both have been used in the largest American bridges built in the twenty years since about 1910. There are bridges in existence, built more than 100 years ago, of both types, and there is no evidence to show that when properly proportioned and adequately maintained, either type, as a type, is not safe and will not endure for an indefinite time. Both types have been advocated and proposed for long-span bridges by engineers of prominent standing, and both types are being manufactured by American manufacturers of the highest reputation and long experience.

Finally, those who would unqualifiedly condemn the eye-bar cable should consider the fact that any wire cable is in reality a combination of wire cable and eye-bar chain, because for structural reasons all the important cables

thus far built terminate in eye-bar chains which differ in no essential element from those which would constitute the main cables.

The cost depends very largely upon circumstances. Early studies for the George Washington Bridge, based upon equitable unit stresses and design details, and upon prices which were then established and indicated by information from the manufacturers, showed plainly that competition between the two types might result in a considerable saving in favor of the eye-bar; and, indeed, if one examines the bids and considers the unit prices which prevailed prior to the competitive bidding one is forced to conclude that this assumption was justified.

A feature which in this particular case favored the economy of the eye-bar cable is the short, steep, back-stays which resulted from the peculiar arrangement of the spans. Such a condition involves a considerable excess section in the wire cable by reason of the practical necessity of making the cable section uniform throughout, while the section of the eye-bar cable can be varied with the stress.

It was suggested that the back-stays be made longer and that that they be given an inclination equivalent to that of the center span cable at the tower. This would have resulted in a materially greater cost for either type and would have made a monstrous looking structure.

The general design and arrangement of the cable anchorages resulted largely from the given topography and geological formations. On the New Jersey side, the hard compact basalt or "trap rock" formation of the Palisades offered, as the logical and most economical solution, inclined anchorage tunnels driven into the rock formation below the floor and refilled with concrete after the placing of the steel anchorage structures to which the cables are attached. On the New York side, the point of intersection of the cables and floor is approximately 100 ft. above the rock surface in Fort Washington Park. A continuation of the cables below the floor and their anchorage in the rock structure was considered, but was found to be neither economical, nor æsthetically desirable, and the anchorage of the cables in a masonry block built up from the ground to the bridge floor was determined upon as the best solution.

This block, although in itself a huge and massive looking structure, forms also a natural abutment for the great arch over Riverside Drive and blends well with the surrounding landscape. Its appearance will be enhanced by appropriate architectural treatment of its surfaces.

The Towers

There is no part of the design of the George Washington Bridge which has called forth as much comment, favorable and unfavorable, on the part of engineers, architects, and laymen, as the towers. Indeed, as the writer has endeavored to show, the design of the suspended structure, the floor, and the cables, resolved itself largely in the application of natural and most simple structural forms which neither required nor permitted architectural treatment to satisfy æsthetics.

The design of the towers, however, is not so well defined. There are widely different meritorious forms and the effect of the towers upon the appearance of the entire structure is perhaps more pronounced than that of any other part. They may enhance or destroy the natural beauty of a graceful suspended structure. There are existing examples which illustrate both effects.

It is futile to theorize about this question. It is largely a matter of æsthetic conception, which is so intensely individual and changeable; nor can it be dealt with on general principles without regard to the local scenery or landscape. Moreover, the æsthetic treatment of a bridge, as that of any other engineering structure, is not always satisfactorily solved even by correct and honest application of engineering principles. The appearance of a structure so conceived may sometimes be materially enhanced by the addition or the architectural embellishment of certain structural parts, whether structurally required or not. The flanking abuments of an arch bridge, and the towers and the anchorages of a suspension bridge, offer opportunity for such enhancement.

At the time the design for the George Washington Bridge was conceived a general tendency prevailed to design the towers of large suspension bridges as steel frames or bents, slender as seen in the elevation of the bridge, with fixed base, but with sufficient flexibility to stand bending resulting from the longitudinal motion of the tower tops to which the cables are fixed.

This type, first successfully applied and architecturally well conceived in the case of the Manhattan Bridge, undoubtedly answers engineering requirements best where great flexibility is required in the case of long side spans, and in many localities (if well designed) it fully satisfies æsthetic requirements. Unfortunately, in a number of cases, such towers have been so crudely designed as to be responsible for much of the adverse criticism of steel towers in general on the part of the art-loving public.

There are existing towers of this kind which are manifestly too slender. Flexibility is not a virtue where it is not needed. Excessive flexibility may cause high secondary stresses which are apt to be ignored, and extremely slender towers give an impression of weakness, but no matter how well designed such slender steel towers may be, and how much they may be justified in certain cases, they can not compare in their monumental effect upon the entire structure with the massive towers so admirably exemplified in the Brooklyn Bridge and in many of the older suspension bridges.

The Brooklyn Bridge, long since its completion surpassed in size and technical achievement, still retains a world-wide reputation as the most fascinating and outstanding structure of its kind, and there can be little doubt that this is due to its admirable gracefulness, coupled with the monumentally conceived towers.

Is it surprising then that in spite of the present tendency toward purely utilitarian and so-called scientifically correct structural forms, repeated efforts should be made on the part of engineers and architects (and such efforts will continue to be made), to produce the effect of massive towers by designs adapted to the use of present-day available materials of construction.

In the case of the George Washington Bridge—owing to its location in a landscape which it is hoped will forever retain its natural beauty, with a background of massive cliffs on one side and the rocky and wooded promontory forming Fort Washington Park on the other—massive looking towers for the bridge appeared particularly well adapted.

As to the adaptability and economy of materials available to produce massive towers of such great height, bridge builders now have, in the combination of steel and concrete, a means of producing much greater strength at much less cost than would be possible with the formerly available stone masonry. This form of construction has made such successful inroads in the field of long-span arches, that its economical application to towers of great height appears logical and feasible.

In fact, comparative studies of steel towers and massive towers made for the George Washington Bridge indicated that at prevailing prices a massive tower of steel and concrete can be designed that will compare very favorably with bare steel towers. A study made by the writer for a slender steel tower (Fig. 21) was given serious consideration in comparison with a massive tower, but, while it might have been a fairly satisfactory sclution, it was the unanimous opinion of those responsible for the design, that in this case it was less meritorious from the æsthetic point of view than the design for the massive tower eventually adopted.

In line with the plan to construct the bridge substantially in two stages of traffic capacity, and to effect the maximum economy, the composite tower was designed as a self-contained steel frame or skeleton which would support itself and, at least, the initial dead load of the suspended structure, but would be encased and strengthened by concrete properly bonded to the steel skeleton and reinforced with reinforcement steel, to obtain the full ultimate carrying capacity of the bridge.

Owing to the great height and comparative slenderness of the massive towers as designed, and the relatively short side spans which govern the longitudinal motion of the cables at their bearings over the towers, it was found feasible, without subjecting the tower to undue stresses from bending, to fix the cable bearings to the towers. Thus, it was possible to avoid movable bearings which, if designed so as to be permanently effective, would be very expensive.

In the course of the final studies for the towers, differences of opinion arose as to the ultimate strength of the combined steel and concrete structure. In order to avoid any controversy about this question and to permit the postponement of the final decision regarding the design without delay to the construction of the steel skeleton, it was decided finally to proportion the steel skeleton so that it would be capable of carrying the entire ultimate load without reliance upon the concrete encasement or else to permit the construction of an independent concrete shell around the steel skeleton if ultimate studies should prove such a solution to be justified. This strengthening of the steel skeleton involved an additional initial expenditure of approximately $800 000.

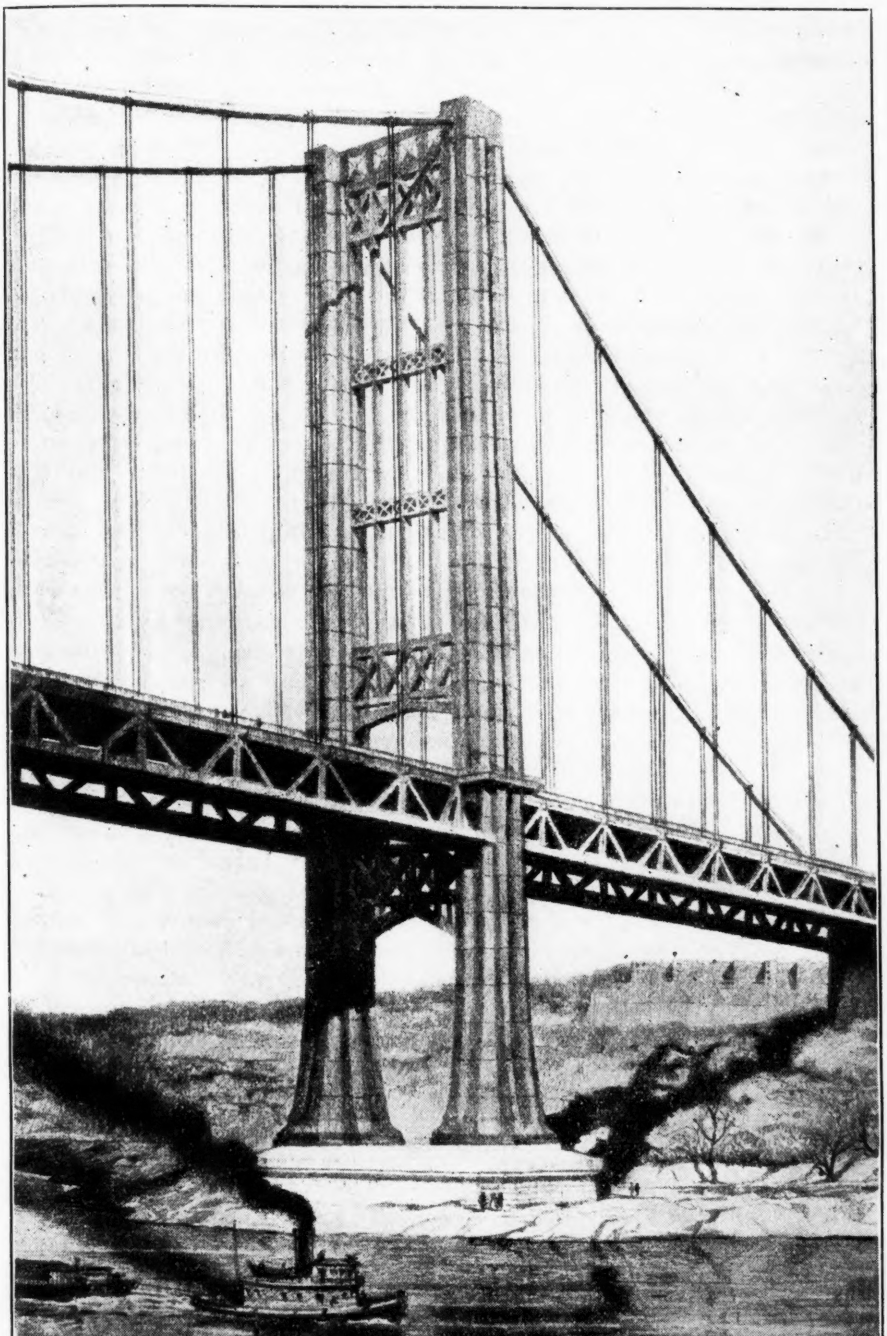


FIG. 21.—STUDY OF SLENDER TYPE OF STEEL TOWER.

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Although no definite detail plans have been developed for the encasement, the towers have been the subject of extended architectural study. In their latest development in that respect (1931) they are illustrated in Fig. 22.

Since the steel tower frame is proportioned to carry the entire ultimate load for which the bridge is designed, the future addition of the concrete encasement, or possibly of a mere shell surrounding the steel frame, as has been suggested, or a combination of both, becomes essentially an æsthetic and architectural question. Before taking any action in this matter, the Port Authority will undoubtedly want to scan public opinion carefully, and particularly give due regard to the attitude of those civic and governmental organizations that are interested in the æsthetic development of the community and the preservation of the beauty of the landscape.

The writer, who has conceived and is primarily responsible for the type and general form of the design, considers the steel towers as they stand to represent as good a design as may be produced by a slender steel bent, and that they lend the entire structure a much more satisfactory appearance than he (and perhaps any one connected with the design), had anticipated. Nevertheless, he believes that the appearance of the towers would be materially enhanced by an encasement with an architectural treatment, such as that developed by the architect, Mr. Cass Gilbert, as illustrated in Fig. 22.

The writer is not impressed by the criticism, based solely on theoretical and utilitarian grounds, that the encasement would constitute a camouflage which would hide the true structure and its function. The covering of the steel frames does not alter or deny their purpose any more than the exterior walls and architectural trimmings destroy the function of the hidden steel skeleton of a modern skyscraper, except to the uninitiated.

Camouflage in this sense would condemn many of the creations in private and public life. It is an essential manifestation of civilization and is not incompatible with sincerity and honesty of endeavor, because an essential part of human effort is to create an æsthetic atmosphere, the value of which cannot be expressed in economic terms. This is evidenced in the craving for beautiful homes and public institutions which yields only to the limits of available means. Why should not a supreme effort be made in that respect in engineering structures, especially those which are viewed daily by thousands or millions of people?

Nevertheless if the encasement should not be built the writer will be satisfied that the effort to produce a massive structure has not been without fruits. The steel tower as it stands owes its good appearance largely to its sturdy proportions and the well-balanced distribution of steel in the columns and bracing.

At its present stage (1932) the top of the steel tower (Fig. 23) requires certain finishing additions. The cable bearings must be housed. It is also recognized that the tops of the towers more than 600 ft. above the water, which offer splendid views of the landscape for many miles all around, should be made accessible to the public by the provision of suitable elevators and protected observation platforms. The artist's sketch shown in Fig. 23

indicates the manner in which this may be accomplished by structural arrangement in harmony with the remainder of the tower frame.

As far as the engineering problem of designing a tower of this height—composed of a steel skeleton and concrete encasement with possible stone facing—is concerned, that presents a number of interesting phases which unquestionably require further research and intensive design study. However, such progress has been made with this composite type of construction, principally in connection with massive arches which exceed in proportions, magnitude, and complexity of stress action those of the proposed towers, that a satisfactory solution for the latter appears to be well within present possibilities. Joint action can be secured largely by proper structural bond between the steel members and the concrete, but probably the most important, and as yet insufficiently clarified, question involved, is that of the relative distribution of stress between steel and concrete and the elastic and non-elastic behavior of these materials under working loads, as well as close to the ultimate strength of the composite structure.

The Tower Steel Frame.—The design of the tower steel frame presented interesting and unusual phases. In its general proportions and outlines it was naturally governed by the design of the massive or encased towers, but an effort was made to design the steel frame so that it would present a neat appearance before its encasement.

In the structural arrangement it was found advantageous, largely on account of the enormous unprecedented load concentrations, to compose the steel structure of simple integral parts of relatively moderate size, but in such a manner as to assure a clear and certain, not necessarily statically determinate, stress action. An arch-shaped trussed frame, with a relatively large number of columns, offered itself in these respects as far more suitable than the usual, seemingly determinate, four-column tower with bracing in four planes.

As designed, the arched steel frame consists essentially of sixteen individual columns arranged in four arch-shaped transverse bents, each having two interior and two exterior columns. The exterior columns of each bent, and the bents themselves are slightly inclined, corresponding to the batter of the tower faces.

The cable bearings or saddles on the tops of the towers are above the two inner pairs of columns, but the load they transmit is distributed to all columns by cross-girders on top and the transverse bracing, in a manner which permits definite distribution of the load and practically exact proportioning of the various column sections. The design involved comparatively slight excess of metal in only the upper sections of the exterior columns.

That the criticism of eccentric loading on top with respect to some of the columns and the so-called static indeterminateness of the entire frame has no justification whatever has been demonstrated clearly by the elaborate stress investigations, supplemented by measurements on a celluloid model. Comprehensive stress measurements on the steel towers substantiated the assumptions and conclusions of the stress investigations.

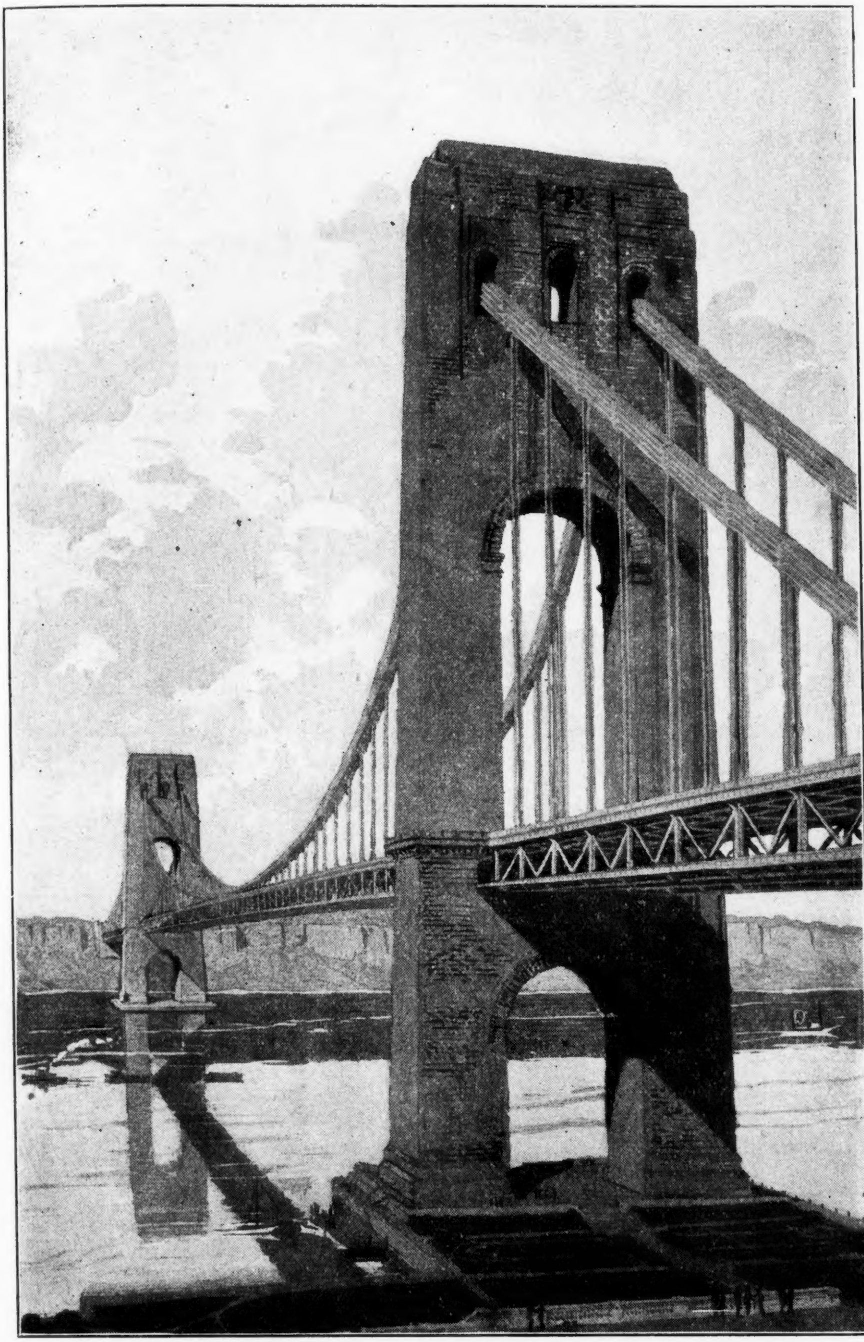

FIG. 22.—ARCHITECTURAL STUDY OF ENCASED STEEL TOWERS, WITH GRANITE FACING.

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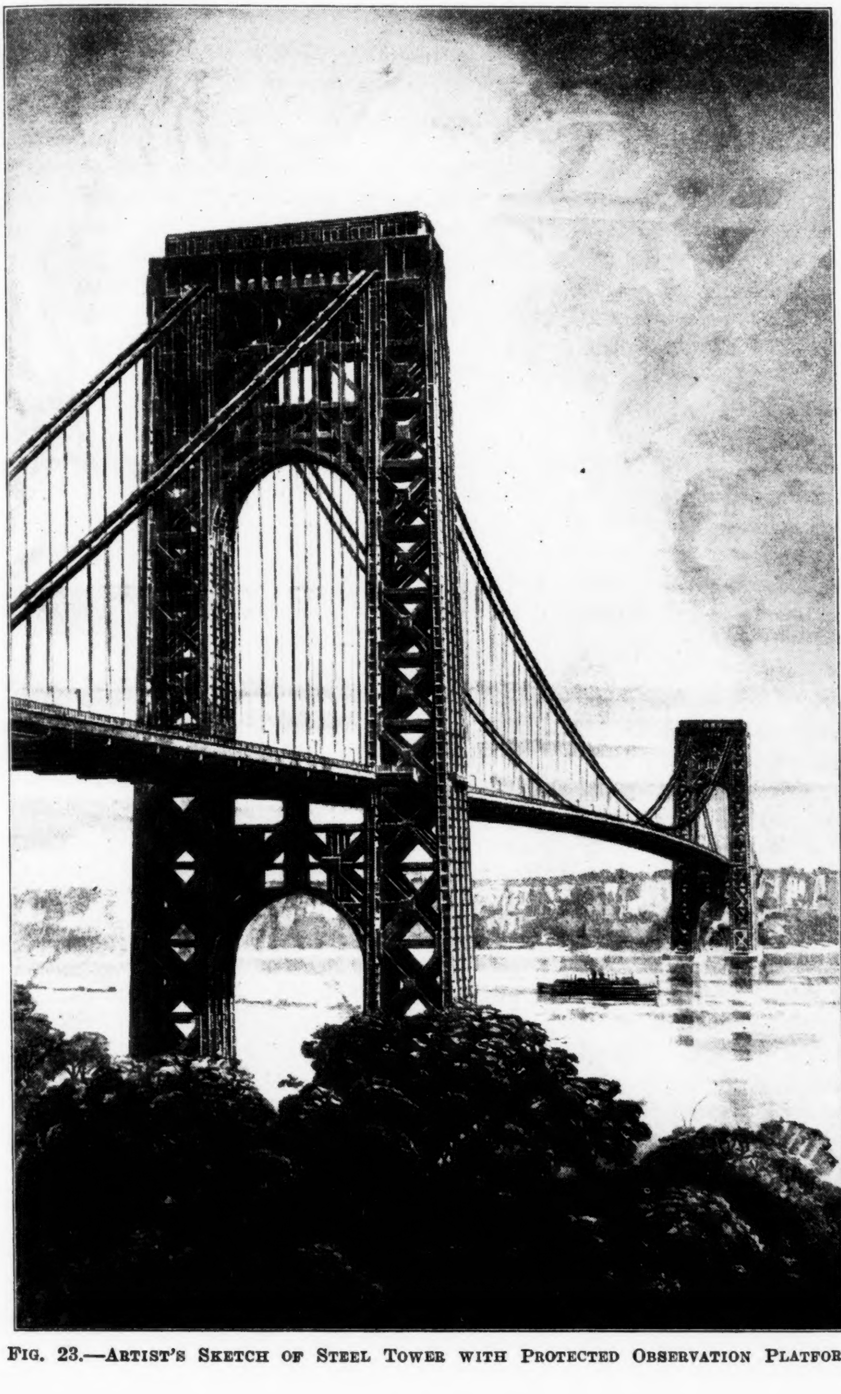

FIG. 23.—ARTIST'S SKETCH OF STEEL TOWER WITH PROTECTED OBSERVATION PLATFORM.

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The Approaches and Highway Connections

Although structurally simple, the planning of the approaches of a large modern highway bridge in a developed community is one of the most intricate problems which such a project presents. Conceptions as to how such approaches should be planned—like those with respect to the planning of highways—have undergone radical changes as a result of the revolutionary change in the character, volume, and speed of highway traffic since the advent of the motor vehicle and, in this respect, engineers have not yet reached the stage of working with generally accepted principles and practices.

Moreover, the planning of approaches to such an important new crossing is intimately tied up with the planning of new, or the improvement of existing, highways leading to and from the crossing, in which many interests are involved. Consequently, the planning of the approaches to the George Washington Bridge was a task of extensive and intensive study and gradual evolution for a period of more than three years.

Under the statutes authorizing the Port Authority to build the bridge, provision is made for the approval of the approach plans by the municipalities in which the approaches are located, as well as by the Governors of the respective States.

As a practical procedure, the Port Authority sought and received close co-operation on the part of the City Government in New York and on the part of the local municipality, the Borough of Fort Lee, and the State Highway Commission in New Jersey, in the early development of the plans and in the necessarily lengthy negotiations which led to their final adoption and approval in the form of agreements between the Port Authority and these governmental bodies. In New Jersey, connections with county roads also involved dealings with the Bergen County authorities.

On both sides of the river the approaches as built are more elaborate and more efficient, as well as more costly, than was originally contemplated. The Port Authority did not shrink from assuming any reasonable additional expenditure for approach facilities as long as they were justified by increased efficiency of traffic distribution and satisfied the Municipal and State Governments.

As the planning proceeded it became apparent that certain principles to meet the requirements of modern traffic would have to be set up and, as far as reasonably practicable, followed. Foremost among these was the avoidance of crossing of traffic lanes at grade, not only on the bridge and approaches proper, but at the points of convergence and divergence of bridge and street traffic, because such crossings invite accidents and retard flow of traffic.

Likewise, primary importance was given to establishing direct connection of the bridge approach with a sufficient number of important streets or highways, which would permit of an adequate distribution of bridge traffic and would avoid excessive concentration in any one artery. Such a system of highway connections of course, had also to have ample flexibility to allow for the unavoidable fluctuations in the traffic flow. The old-time idea of a

single bridge plaza into which and from which all distribution takes place is no longer adapted to the modern requirements of speed and safety and must be superseded by a more efficient, but also more costly, system of direct roadway connections with the important highways carrying traffic to and from the crossing.

The adoption of conservative grades and curvatures and of ample width of approach roadways to secure greater safety and to permit more rapid flow of traffic also was considered essential.

The New Jersey Approach.—The general layout of the approaches is shown in Fig. 24. The roadway or upper floor of the bridge strikes the face of the Palisades at an elevation approximately 40 ft. below the top of the cliff; but the latter slopes down toward the west and meets the roadway grade about 600 ft. west of the face of the cliffs. Consequently, it became necessary to build the New Jersey Approach immediately west of the face of the cliffs in a rock cut.

Hudson Terrace, the most easterly north-and-south artery, lies in a depression and could be bridged over by the approach viaduct without material changes in its grade. It is connected, however, with the bridge approach both on its easterly and westerly sides by separate ramps. West of Hudson Terrace the main bridge ramp strikes the existing ground surface and widens out to form a plaza about 450 ft. long by 200 ft. wide, on which the toll facilities are located.

From this plaza roadway ramps ascend westerly to Lemoine Avenue (New Jersey State Highway Route No. S-1-A), while the central portion of the approach descends and passes under Lemoine Avenue and thence continues as a depressed roadway about 2 000 ft. to the west, where it connects with State Highways Nos. 1, 4, and 6, and a proposed county road. The plaza also permits direct connection with the local streets north and south and with Hudson Terrace to the east. The depressed roadway west of Lemoine Avenue also provides direct connection with Center Avenue, an important local artery to the west of Lemoine Avenue.

Thus, the bridge approach connects directly with seven more or less important through arteries and with a number of local streets, and all the important connections permit of uninterrupted flow of traffic without crossing of lanes at grade.

The New York Approach.—The New York Approach consists essentially of an elevated approach ramp from the anchorage in Fort Washington Park to a surface and a sub-surface plaza immediately west of Fort Washington Avenue, and of direct highway connections from there to Riverside Drive, Fort Washington Avenue, Broadway, and Amsterdam Avenue.

In accordance with the agreement between the City of New York and The Port of New York Authority, this approach was planned to be constructed in stages. In the initial stage of construction, the main approach ramp from the anchorage in Fort Washington Park is only 60 ft. wide and comes to grade at Northern Avenue. From there, side ramps connect easterly with Fort Washington Avenue, while the central roadway descends easterly to a

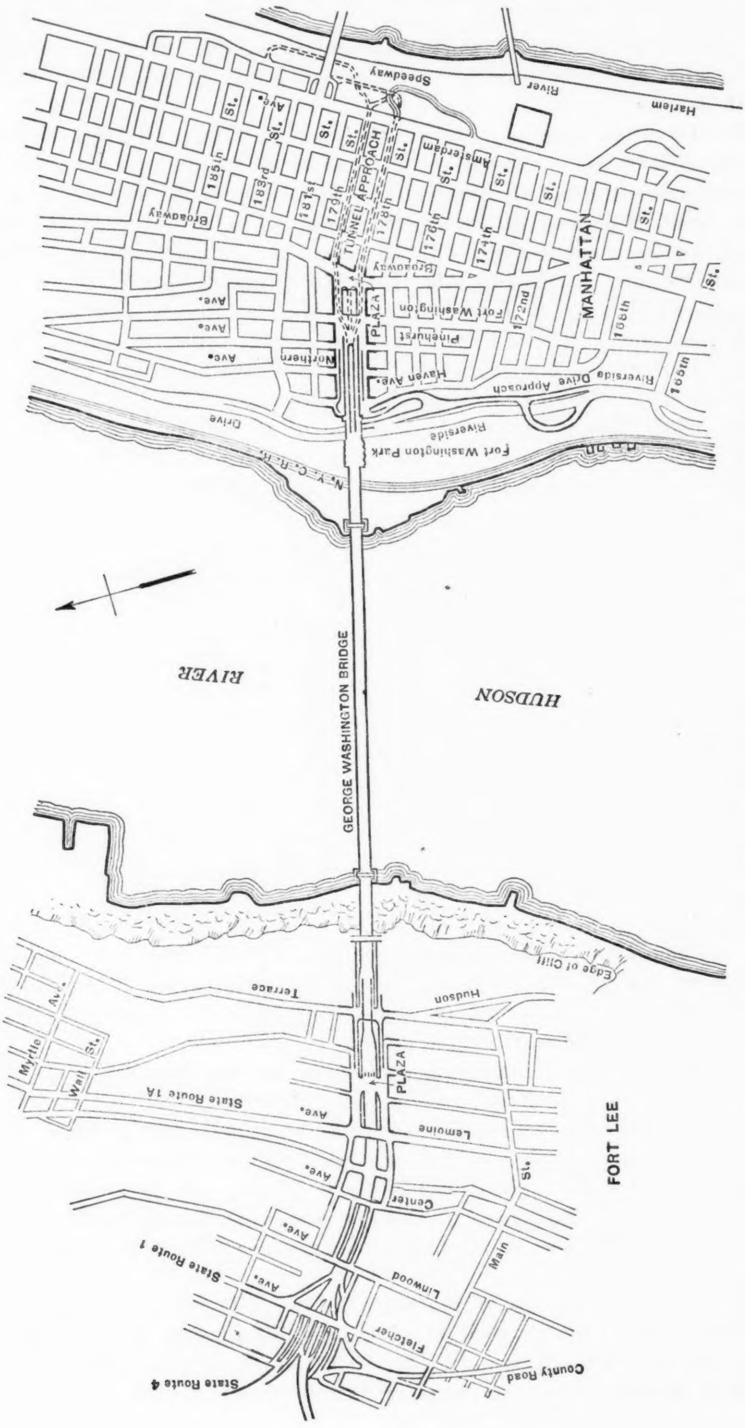

FIG. 24.—LOCATION MAP OF GEORGE WASHINGTON BRIDGE AND HIGHWAY APPROACHES.

sub-surface plaza west of Fort Washington Avenue, where connections are made with vehicular tunnels, each for two lanes, in West 178th and West 179th Streets.

In the initial plan the surface roadways in 178th and 179th Streets east of Fort Washington Avenue are widened to 36 ft. by reducing the width of the sidewalks in order to accommodate the bridge traffic which may go to, or come from, Broadway and adjacent streets. The initial plan also provides for the roadway connections with Riverside Drive, which run from Northern Avenue westerly parallel to 178th and 179th Streets north and south of the main approach ramp to Haven Avenue. West of Haven Avenue these roadways turn south, the on-bound roadway passing under the main approach ramp. Both the off-bound and on-bound roadways branch to permit connections on the east and the west side of Riverside Drive. The roadway for bridge traffic to and from the west side of the Drive is carried across it over an arch of 120-ft. span, approximately opposite West 171st Street. These connections permit trans-Hudson traffic to pass directly to and from Riverside Drive both northbound or southbound without crossing traffic on the Drive.

Toll Collection and Operation Facilities.—Owing to the necessity of collecting tolls, elaborate facilities therefor, as well as for police regulation, maintenance, and repairs had to be made. Property being much less expensive for such facilities on the New Jersey side and the concentration of these administrative activities being desirable, provision for them was made on the New Jersey side only, except in so far as traffic control, more particularly in the tunnnels, may be necessary on the New York side.

Facilities for the collection of tolls are located on the spacious plaza formed by the widening of the bridge ramp west of Hudson Terrace and on the two side ramps connecting the main approach roadway with the east side of Hudson Terrace. The plaza width is sufficient to permit the use of sixteen lanes at the toll booths. These are arranged in pairs and are flanked on either side by a toll-house.

The dignified architectural treatment of the toll buildings is in keeping with the character of the bridge structure. This is secured by extending a continuous canopy over the line of toll booths terminating at the toll-houses. Aluminum surfacing is used on the booths while the toll-houses are two-story granite buildings of simple design.

The most advanced automatic equipment for the registration of vehicles and for the recording of collections is installed at all toll lanes. The type of vehicle and fare collected are recorded automatically and, at the same time, are indicated upon a dial system mounted at the toll booth. Beyond the toll booth a treadle records the passage of the car, thus providing an additional check. The pair of toll booths at the foot of each of the two ramps east of Hudson Terrace are similar in design and equipment to those at the plaza.

The adequate lighting of the plaza in the toll-collection area presented a special problem which was solved by the installation of four flood-light towers which provide illumination for 80 000 sq. ft. of plaza (200 ft. in width by 400 ft. in length) leaving the plaza entirely free of obstruction.

A field office and garage for the local administration of the bridge is located immediately south of the plaza. It is a two-story structure of colonial architecture with stone walls trimmed with granite. It provides suitable facilities for the Superintendent, his assistants, a maintenance supervisor, clerks, police officers, and a physician. It likewise provides locker space for the police and maintenance force, and space for store rooms and a machine shop, the latter in connection with a garage of 16-car capacity which is attached to the field office.

Cost of Bridge and Approaches

At the time the bridge was financed in December, 1926, it was estimated that initially—ready for a four-lane vehicular traffic, but with provision as far as necessary for an ultimate capacity of eight vehicular lanes and four rapid transit tracks—the bridge would cost approximately $60 000 000. This included the highway connections on the New Jersey side to Hudson Terrace and Lemoine Avenue, and on the New York side the complete main approach for full capacity and connections with Fort Washington Avenue and Broadway, a total length of bridge and approaches of 8 720 ft.

As a result of negotiations with the City of New York there were subsequently added the highway connection with Riverside Drive, utilizing partly existing city streets, and the vehicular tunnel approach in 178th and 179th Streets between Amsterdam Avenue and Fort Washington Avenue. The 178th Street Tunnel was to be built initially and that in 179th Street later. In order to offset this additional initial expenditure partly, it was decided to omit, initially, the side ramps of the main approach west of Fort Washington Avenue and also the extension of the approach to Broadway. In part, the more costly additions were made feasible, without securing additional funds, owing to the fact that the estimates made in 1926 proved to be generally high, coupled with a decided lowering of costs in 1930 and 1931, as a result of the continued business depression.

The actual initial cost of the project, as now carried out and with ample allowance for the granite encasement of the New York anchorage, for the completion of the 178th Street Tunnel, completion of the top of the steel towers, and for miscellaneous other work yet to be done as part of the initial stage, is approximately, as follows:

Bridge Proper, Inclusive of Anchorages:	
Construction, engineering, and administration	$31 329 000
Real estate and easements	200 000
Total initial cost of bridge proper	$31 529 000
New York Approach, Inclusive of Connections with Amsterdam Avenue and Riverside Drive:	
Construction, engineering, and administration	$6 372 000
Real estate and easements	8 882 000
Total initial cost of New York Approach and highway connections	$15 254 000

New Jersey Approach, Inclusive of Connections with Hudson Terrace and Lemoine Avenue:	
Construction, engineering, and administration..........	$2 418 000
Real estate and easements..........................	1 113 000
Total initial cost of New Jersey Approach and highway connections	$3 531 000

Summary of Cost of Bridge, Approaches, and Highway Connections:	
Construction, engineering, and administration..........	$40 119 000
Real estate and easements..........................	10 195 000
Total construction and rights of way................	$50 314 000
Interest during construction........................	4 543 000
Total initial cost of project, inclusive of interest during construction	$54 857 000

In the comparison of this cost with the par value of the outstanding bonds ($50 000 000) plus the advances by the two States ($10 000 000), there must be added to the cost an item of $3 030 000 for discount on the bonds, bringing the total initial cost of the project to $57 887 000.

In the original $60 000 000 estimate of 1926, the discount on bonds was not included, it having been assumed that any discount or premium would be a matter of financing operation, depending largely on market conditions and the interest rate on the bonds, and might properly be distributed over the period of amortization of the bonds. However, it was decided later to be desirable to amortize this item entirely out of construction funds.

The foregoing costs of construction, engineering, and administration of the bridge proper is made up of the following principal items:

Cost of Bridge Proper:	
Pier foundations and borings..........................	$1 275 000
Anchorages (exclusive of steel).......................	2 730 000
Anchorage steel	1 467 000
Steel towers	8 069 000
Cables and suspenders.................................	10 726 000
Steel floor system (highway deck only)................	2 679 000
Floor-slab, railings, and miscellaneous construction......	957 000
Electrical equipment and installation.................	85 000
Miscellaneous and contingencies.......................	450 000
Engineering and administration........................	2 891 000
Total initial cost of construction, engineering, and administration of bridge proper.................	$31 329 000

An estimate made to determine what the cost would have been in case the bridge had been designed and built with only one deck for a six-lane roadway and two sidewalks, without provision for rapid transit traffic shows that the reduction would have been $15 000 000 and the total initial cost without discount on bonds, therefore, approximately $40 000 000.

About 95% of this possible reduction in cost is due to provisions for the greater capacity in the bridge proper, and only about 5% for such provisions made in the approaches. The vehicular tunnel in 178th Street is included in the aforementioned cost for a six-lane bridge.

Progress and Principal Construction Contracts.— As hereinbefore mentioned, the first Port Authority bonds for this project were sold on December 9, 1926. It was at that time estimated that the bridge would be completed ready for vehicular traffic on or about July 1, 1932.

Final borings at the site of the New Jersey Tower were then already under way under a contract with the Osborne Drilling Company, and on May 19, 1927, the first construction contract was let.

The major construction contracts were signed in accordance with the following schedule:

May 19, 1927.—Foundation for the New Jersey Tower, Contractor, Silas B. Mason, Incorporated, approximately $1 059 000.

June 18, 1927.—Excavation for the New Jersey Anchorage and Approach, Contractor, Foley Brothers, Incorporated, approximately $930 000.

November 4, 1927.—Steel towers and floor system, Contractor, McClintic-Marshall Company, approximately $10 753 000.

November 4, 1927.—Cables, suspenders, and anchorage steel work, Contractor, John A. Roebling's Sons Company, approximately $12 193 000.

May 4, 1928.—New York Anchorage and Tower Foundation, Contractor, Arthur McMullen Company, approximately $1 088 000.

December 6, 1929.—Demolition and removal of buildings on site of the New York Approach, Contractor, Klosk Contracting Company, approximately $150 000.

July 3, 1930.—Miscellaneous construction for New Jersey Approach at Hudson Terrace, Contractor, George M. Brewster and Son, approximately $323 000.

August 7, 1930.—Main approach ramp of the New York Approach, Contractor, Cornell Contracting Corporation, approximately $870 000.

August 7, 1930.—Vehicular tunnel in West 178th Street of the New York Approach, Contractor, Cornell Contracting Corporation, approximately $2 126 000.

October 14, 1930.—Riverside Drive connection of the New York Approach, Contractor, William P. McGarry Company, approximately $1 200 000.

February 2, 1931.—Paving and miscellaneous construction of the New Jersey Approach, Contractor, George M. Brewster and Son, approximately $565 000.

March 20, 1931.—Paving, railings, and miscellaneous construction work for the Main Bridge and New York Anchorage, Contractor, Corbetta Concrete Corporation, approximately $495 000.

Miscellaneous contracts were let in 1931 for the field office building, toll booths, electrical installations, flood-light towers, building alterations, and final painting to the aggregate amount of approximately $603 000.

Progress of construction work was so favorable as to make it possible to open the bridge for vehicular traffic on October 25, 1931, or about eight months in advance of the original schedule.

The actual period of construction from the date of financing to the opening of the bridge to traffic, therefore, consumed about a month less than five years, and from the date of the first construction contract less than four and one-half years.

Formal ground-breaking ceremonies took place on September 21, 1927. The raising of the first foot-bridge rope into place was celebrated on July 9, 1929, and the opening of the bridge was preceded by elaborate ceremonies on October 24, 1931.

Organization and Personnel

The Port of New York Authority, which under mandate from the States of New York and New Jersey has built, owns, and operates the George Washington Bridge is composed of twelve Commissioners, six from each State, appointed by the respective Governors. The Commissioners are men of broad and varied experience in business and public affairs. They serve without compensation.

During the construction of the George Washington Bridge, the following acted, successively, as Chairmen of the Port Authority: Mr. Julian A. Gregory (November 19, 1924, to May 20, 1926), former Governor of New Jersey, George S. Silzer (May 27, 1926, to June 30, 1928), and Mr. John F. Galvin (July 12, 1928, to date). The other Commissioners in office during that period include: Messrs. Ira R. Crouse, Howard S. Cullman, George R. Dyer, Frank C. Ferguson, William C. Heppenheimer, George deB. Keim, John F. Murray, John J. Pulleyn, Schuyler N. Rice, Alexander J. Shamberg, Herbert K. Twitchell (deceased), and Joseph G. Wright. Mr. Shamberg had been one of the Commissioners of the former New York Interstate Bridge Commission (later, the New York State Bridge and Terminal Commission), since its creation in 1906, and General Dyer had served in that Commission since 1909 and was its Chairman during the construction of the Holland Tunnel.

The executive and administrative functions, including the financial and real estate transactions, are centered in the General Manager. Throughout the period of construction this responsible office has been held by Mr. John E. Ramsey.

The many complicated and unprecedented legal and legislative matters have been in charge of the Legal Department of the Port Authority, headed by Mr. Julius Henry Cohen, as General Counsel.

The engineering matters have been attended to by the Engineering Department, under the direction of the writer as Chief Engineer. He had the very able assistance of the following principal members of his staff:

Edward W. Stearns, M. Am. Soc. C. E., as Assistant Chief Engineer, attended generally to administrative functions in the Department and the preparation of contracts and specifications. Allston Dana, M. Am. Soc. C. E., as Engineer of Design, has been in charge of the Design Division. John C. Evans, Assoc. M. Am. Soc. C. E., Terminal Engineer, attended to

the general studies for the approaches and highway connections. Montgomery B. Case, M. Am. Soc. C. E., Engineer of Construction, headed the Construction Division which made all field surveys and directed and supervised the work in the field.

The architectural studies were made by Cass Gilbert, Architect, in collaboration with the Staff.

The Chief Engineer also had the advice of the following Consulting Engineers: William H. Burr, Gen. George W. Goethals (deceased), Leon S. Moisseiff (on design), Daniel E. Moran (on foundations), Ole Singstad (on vehicular tunnel approach), and Lewis B. Stillwell (on electric installations), Members Am. Soc. C. E., and Mr. Joseph B. Strauss.

Gustav Lindenthal, Hon. M. Am. Soc. C. E., as Consulting Engineer, rendered special advice on design questions.

Charles P. Berkey, M. Am. Soc. C. E., as Consulting Geologist, made the geological investigations.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

FORESTS AND STREAM FLOW

By W. G. Hoyt,[1] M. Am. Soc. C. E., and H. C. Troxell,[2] Assoc. M. Am. Soc. C. E.

Synopsis

The effect on stream flow of changes in forest and brush cover on specific natural areas is described in this paper. An experiment by the United State Forest Service and the United States Weather Bureau was conducted from 1910 to 1926 on two contiguous tracts of land in Southern Colorado. Data for this study were taken from a report[3] by Messrs. C. G. Bates and A. J. Henry. Stream flow measurements by the United States Geological Survey in co-operation with the State of California and the County of Los Angeles were begun in 1916 on certain areas in California, on some of which accidental denudation by burning afforded opportunity for comparisons not heretofore published. Detailed observations were made in both these areas for several years before changes in cover were accomplished by cutting and by fire, and were continued for several years after such changes.

The effects of the change in vegetable cover on the stream flow characteristics are discussed in this paper and conclusions drawn.

Introduction

In November, 1908, the late H. M. Chittenden, M. Am. Soc. C. E., then Lieutenant-Colonel, Corps of Engineers, U. S. Army, presented a paper[4] before the Society entitled "Forests and Reservoirs in Their Relation to Stream Flow, with Particular Reference to Navigable Rivers." He summarized the opinions commonly accepted at that time, to the effect that forests exert a beneficial influence on stream flow in the following ways: (1) By

Note.—Presented at the Annual Convention, Yellowstone National Park, July 6, 1932. Written discussion on this paper will be closed in **November, 1932,** *Proceedings*.

[1] Prin. Hydr. Engr., Conservation Branch, U. S. Geological Survey, Washington, D. C.

[2] Associate Hydr. Engr., Water Resources Branch, U. S. Geological Survey, Los Angeles, Calif.

[3] *Monthly Weather Review, Supplement 30.* (Copies of this paper may be obtained from the U. S. Forest Service, Washington, D. C.)

[4] *Transactions*, Am. Soc. C. E., Vol. LXII (1909), pp. 245–546.

storing the waters from rain and melting snow in the bed of humus that develops under forest cover, preventing their rapid rush to the streams and releasing them gradually afterward, thus acting as true reservoirs in equalizing the run-off; (2) by retarding the snow melting in the spring and prolonging the run-off from that source; (3) by increasing precipitation; and, (4) by preventing erosion of the soil on steep slopes and thereby protecting watercourses, canals, reservoirs, and similar works from accumulation of silt.

Colonel Chittenden's study had a negative conclusion. He found that no material influence upon stream flow could be attributed to forests. In its final report,[5] presented January 19, 1916, the Special Committee on Floods and Flood Prevention stated that if reforestation is considered merely from a commercial standpoint, the value to the country of reproducing timber is so great that it obtains general approval, particularly of engineers, who appreciate more than any other class the disadvantages to which the country will be subjected by the destruction of its forests.

Nevertheless, according to the Special Committee, some advocates of reforestation have claimed that forest destruction has increased the height and frequency of floods and diminished the discharge during low water. The basis of those claims for reforestation has been the beneficial influence of forests in preventing floods and improving the navigability of rivers.

This was a moot question in 1916 and the Committee pointed out that even the advocates of reforestation as a means of flood control failed to give any quantitative determination of the effects of forests upon floods. It was stated that, for the engineer to utilize any agency practically, he must know that it will produce a positive effect on which he can rely and which he can measure. The data as to the effect of forestation, soil observation, and kindred methods of flood control were admittedly not susceptible of quantitative analysis at that time.

Expressing his opinion as of 1930, the late Allen Hazen,[6] M. Am. Soc. C. E., called attention to the fact that foresters have urged the importance of forests in controlling and regulating stream flow, increasing the quantities of water available for use, and preventing and reducing flood flows. Although Mr. Hazen felt that the public had generally accepted this point of view, water-works engineers, he declared, have required some proof before they could accept this popular doctrine. Furthermore, according to Mr. Hazen, real evidence of the effect of forests upon stream flow is meager and unsatisfactory, involving the solution of a difficult problem to prove conclusively one way or the other.

He admitted that forests undoubtedly affect stream flow in many ways, but that there was no evidence that they modify major flood flows materially. Quoting:

"There is no reason to think that clearing of land in our country and putting it under cultivation has materially increased the flood quantities over those that existed in previous centuries. Reforesting cleared and burned

[5] *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1218.

[6] "Flood Flows—A Study of Sequences and Magnitude," pp. 152–154, John Wiley & Sons, Inc., 1930.

mountain areas at the head-waters of rivers is advantageous from many aspects, but it is not to be counted on to make a material reduction in floods in rivers flowing from them."

During the twenty years that have elapsed since Colonel Chittenden's paper was written, extensive investigations have been conducted in the Wagonwheel Gap area, in Colorado, for the express purpose of determining the actual effect of forests on stream flow. By chance, also, a forest fire in Southern California destroyed the trees and undergrowth in an area from which records of stream flow had been collected for seven years previously. Information, therefore, is available to show quantitatively the effect on stream flow brought about (*a*) by the complete deforestation of a high mountain area having an annual precipitation of about 20 in. (one-half of which occurs in the form of snow); and (*b*) by the complete denudation of a semi-arid coastal mountain region on which the annual precipitation ranges from 16 to 60 in., with little snow. Conclusive evidence of the effect of forest cover on stream flow from these two widely different areas should be, not only of considerable interest, but also of great value in formulating policies relating to forests and the conservation of water.

It is strange, but nevertheless true, that until the investigations in the Wagonwheel Gap area were completed, no observations had ever been made of the actual effect of deforestation upon stream flow, conducted on a sufficiently large scale and covering a long enough time to insure that the results obtained and the conclusions based on them would not be subject to serious doubt. The investigations near Wagonwheel Gap and in Southern California differ materially from any made heretofore. The observations were made first on the forested areas for a length of time sufficient to determine their natural characteristics; then one of the areas was deforested and one accidentally burned, and observations were continued after the change. In previous investigations, forested and partly deforested areas have been compared only after the change had taken place, and as the regimen of the areas under natural conditions is not known, there is reasonable doubt as to the proper evaluation of the differences in run-off that were observed.

Scope of Investigations

Wagonwheel Gap, Colorado.—Investigations were conducted jointly in the Wagonwheel Gap area, in Colorado, by the United States Forest Service and the United States Weather Bureau, during the period, June 1, 1910, to October 1, 1926.[7] The greatest credit is due to the scientists of the United States Department of Agriculture who conceived the project and so successfully carried on detailed observations. The publication of the basic data affords students and investigators the fullest opportunity to make independent analyses.

With this idea in view the writers have made a study of the basic data and have endeavored to present graphically, and in tabular form, in such a

[7] *Monthly Weather Review, Supplement 30,* entitled "Forest and Stream-Flow Experiment at Wagonwheel Gap, Colorado: Final Report on Completion of the Second Phase of the Experiment," by C. G. Bates, Silviculturist, U. S. Forest Service, and A. J. Henry, Meteorologist, U. S. Weather Bureau, 1928.

way that it may be most easily visualized by engineers and water users, the change in stream flow resulting from deforestation.

Since quantitative measurements of the actual differences in run-off resulting from deforestation and denudation are now available for the first time, the writers feel that the facts should be given far greater weight than conclusions reached by inductive reasoning.

In the following discussion the data collected, both at Wagonwheel Gap and in Southern California, are analyzed to show the effect of deforestation on (1) total run-off and distribution; (2) maximum flow and date of occurrence; (3) summer flow; (4) minimum flow and date of occurrence; and (5) erosion and silt content.

As all the conditions surrounding the Wagonwheel Gap experiment are fully discussed in the report cited, only a general outline of them will be

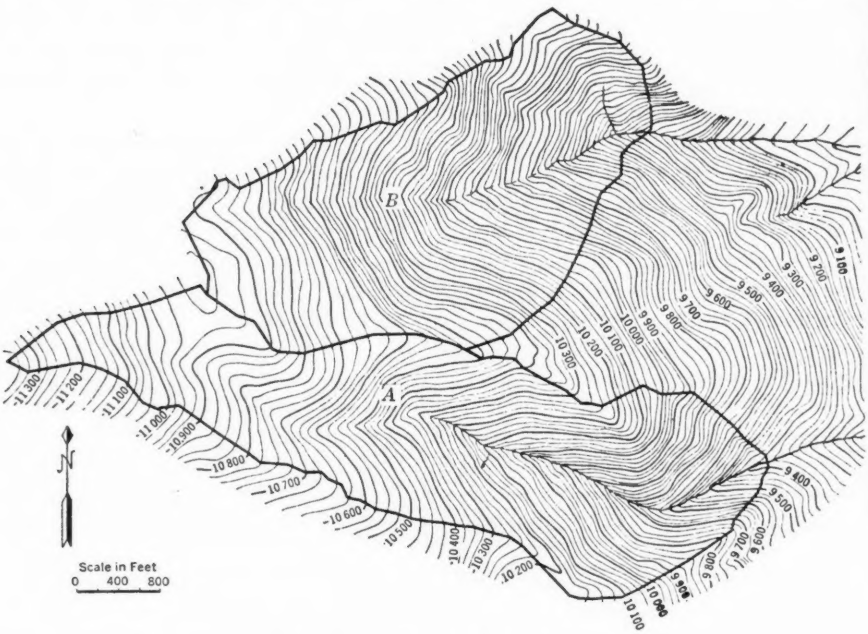

FIG. 1.—AREAS *A* AND *B*, WAGONWHEEL GAP, COLORADO

given in this paper. The areas selected (designated for convenience Area *A* and Area *B*) were two contiguous mountain drainage basins in Mineral County, Southern Colorado, in the Rio Grande drainage basin, at altitudes between 9 000 and 11 000 ft. (see Fig. 1). The forest cover in both areas was representative of the Rocky Mountain region as a whole, and, therefore, other conditions being similar, the results may be reasonably assumed to be representative of many mountain areas throughout the West.

Geologically and topographically both tracts were as similar as it is possible for two small contiguous areas to be, and their combined area

(Area *A*, 222.5 acres, and Area *B*, 200.4 acres) was so small that they were both subject to practically identical meteorological conditions. The measurements of run-off and all meteorological observations were made with the greatest possible precision, and the results obtained may be taken at their face value.

The forest cover at the beginning of the investigation was as shown in Table 1.

TABLE 1.—NATURE AND EXTENT OF FOREST COVER IN WAGONWHEEL GAP, COLORADO

Type of cover	Percentage		Type of cover	Percentage	
	Area *A*	Area *B*		Area *A*	Area *B*
Burned and not restocked (mostly spruce)	9.5	6.6	Aspen with conifers	14.4	17.1
Barren or rock slide	2.7	3.0	Douglas fir	8.8	11.4
Grass	9.4	6.1	Mainly spruce	11.9	12.0
Aspen without conifers	34.3	43.8	Bristlecone pine (open)	9.0	....
Total				100.0	100.0

During the summer of 1919 Area *B* was deforested, all tree growth being cut except in a narrow strip along the course of the stream, which was not cut until 1920. The slash from the larger green conifers and the tops and stems of the smaller evergreens and aspens were burned in 1921. After the deforestation there was a growth of grass, herbs, and aspen, and, in places, the aspen sprouts had reached heights of 3 to 6 ft. at the end of the 7-year period. In general, the grasses were reproduced on south exposures and grass and herbs on north exposures. Messrs. Bates and Henry have used the term "denudation" to describe the removal of the trees from the area. Webster defines denudation as "act of denuding; * * * the laying bare of rocks by the removal of overlying material; erosion." The writers believe that the term "deforestation" more correctly defines the change that occurred on Area *B*, because only in isolated places where slash was burned (comprising a very small percentage of the whole), was there actual destruction of soil covering. "Denudation," however, correctly describes the change that occurred in the Southern California area.

Reliable observations at Wagonwheel Gap were made for eight years in the natural state and for seven years after deforestation. (There is some doubt as to the accuracy of the run-off records prior to August, 1911, so that the complete record runs from September, 1911, to September, 1926.) This study is based necessarily on the hypothesis that during the first eight years the run-off relations and differences between the two areas were so well indicated that the results obtained can be safely used as indices of the changes in run-off resulting from complete deforestation.

Many of the pertinent annual and monthly climatological data for the two areas are given in the accompanying tables. Briefly, the mean annual temperature was about 34° Fahr., the range being from 80° Fahr. during June, July, and August, to temperatures below zero occasionally during all

months from October to March. The mean annual precipitation was about 21 in., one-half of which occurred in the form of snow. The climatic conditions were very similar during both periods.

The mean annual temperature of Area *A* was 34.0° Fahr., during both periods. The mean annual temperature of Area *B* was 1.3° warmer after deforestation than before. The mean annual precipitation over Area *A* was 21.03 in., during the first period and 21.16 in. during the second period; over Area *B*, the mean annual precipitation was 21.10 and 20.83 in., respectively, during the first and second periods. The accumulated precipitation on these two areas generally agrees within 0.5%, or less than the probable error of the observations. In the accompanying tables observations of run-off are given, in inches, over the drainage basin, corresponding to the base data given in the U. S. Weather Bureau report, and, also, in second-feet per square mile.

Southern California Area.—On August 31, 1924, a forest fire started in the San Gabriel Canyon, north of Azusa, Calif. The fire spread rapidly and had soon burned over some of the drainage areas tributary to the San Gabriel River (see stipled area in Fig. 2). Among these areas were those of Fish, Rogers, and Sawpit Creeks. The fire did not reach the drainage basin of Santa Anita Creek. On each of these creeks the U. S. Geological Survey had been operating gauging stations for several years, in co-operation with the California State Engineer and Los Angeles County. Each station was equipped with a water-stage recorder mounted in a concrete house. Measurements of discharge were made with a current meter, and the records were kept and published in the usual manner.

Of the three burned areas that of Fish Creek offers the best discharge record, in that there are no diversions from the stream nor artificial regulation above the gauging station. On Rogers Creek there are two diversions above the gauging station. No record has been kept of the quantity diverted, but although the structures are relatively small, they divert the entire flow during the summer. On Sawpit Creek the City of Monrovia diverts water above the gauging station. The city has a series of tunnels in the head-waters, which is one of the main sources of the flow in the Monrovia pipe line.

For these reasons Fish Creek was selected for the study. To establish the effect of the fire on the discharge of this creek, it was necessary to determine the relation between its flow and that of Santa Anita Creek for the period prior to the fire.

The canyons of both Fish and Santa Anita Creeks are well defined by mountain divides and are separated only by the basin of Sawpit Creek. Both streams originate at altitudes greater than 4 000 ft. The east side of Mount Wilson drains into Santa Anita Creek. Monrovia Hill, a peak of 5 267 ft., separates the head-waters of the two streams, Fish Creek draining the east side and Santa Anita Creek the west. Santa Anita Canyon is fan-shaped; Fish Canyon is oblong; and the side walls of both canyons are steep and rugged. The drainage area above the gauging station on Fish Creek is 6.5 sq. miles, and that of Santa Anita Creek, 10.5 sq. miles. The vegetative cover consisted of a heavy brush growth of sumac (*Rhus sp.*) and mountain mahogany (*Cercocarpus montanus*). Interspersed between the brush growth

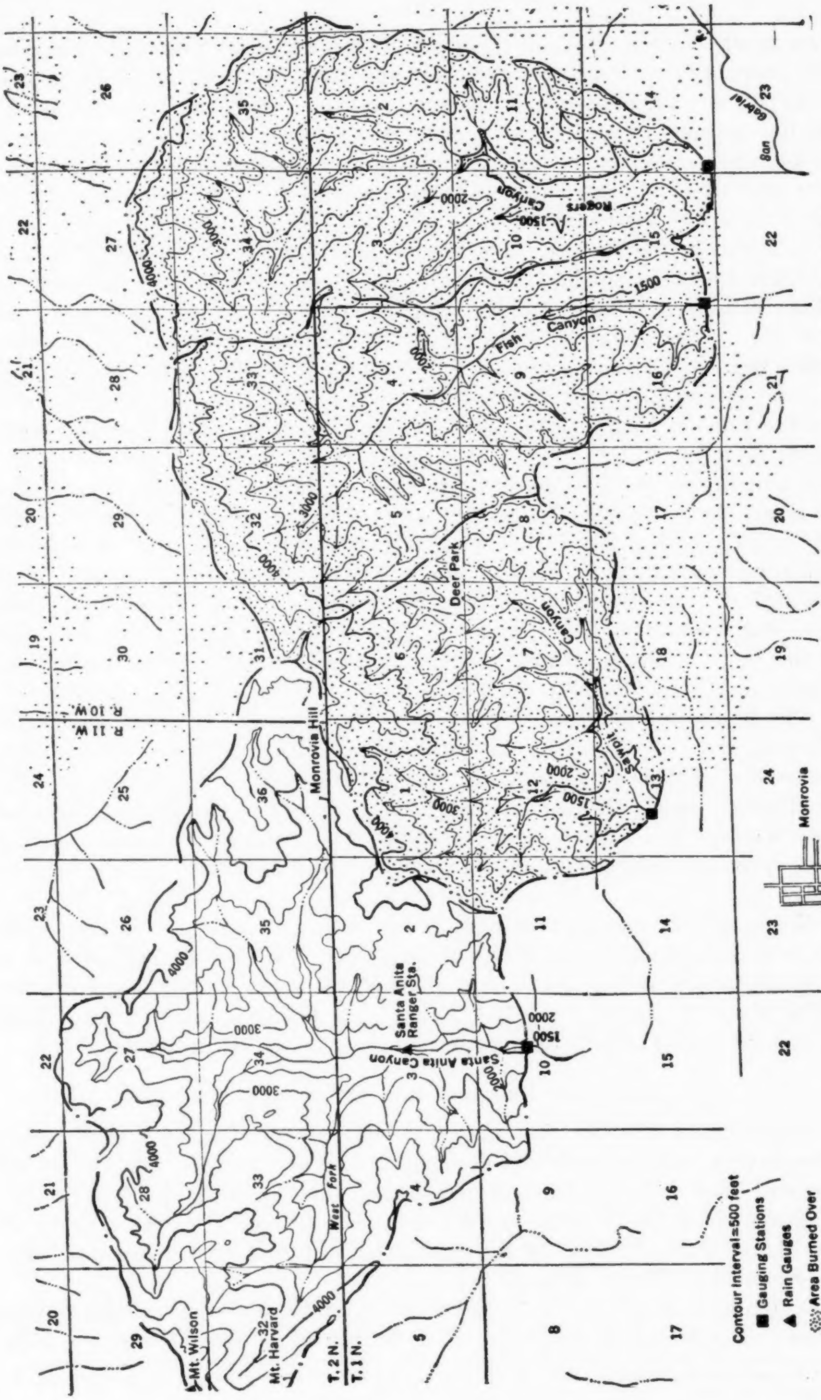

FIG. 2.—SANTA ANITA, FISH, ROGERS, AND SAWPIT CREEKS, IN THE SOUTHERN PART OF CALIFORNIA

were much smaller plants, such as white sage (*Artemisia californica*), grass, and yucca (*Yucca radiosa*). In sheltered tracts and on the north sides of many of the numerous small tributary canyons grew small groups of oak. In the bottoms of the canyons along the streams were large numbers of sycamores and alders. The areas are in the Angeles National Forest, but they support no timber, and neither area was used for grazing. Their principal value was for recreation.

The fire in the summer of 1924 completely stripped the canyon walls of Fish Creek of all plant life and tree litter. Some of the alders and sycamores adjacent to the stream escaped burning through the efforts of the fire fighters. After the first rains in the following spring a different plant association comprising sage, wild flowers, and grasses developed, which tended to cover the soil left bare by the fire. By the fall of 1930 little evidence of the fire remained except that possibly larger areas of white sage and small plants grew in.

The gauging stations on both streams are ideally located. The station on Santa Anita Creek is in a rock gorge and the crest of small waterfalls forms the control. On Fish Creek, the control is an outcrop of bed-rock, which was smoothed up by building a small concrete dam on it. It is doubtful if any underflow passes either station. There are no diversions above either station. The stream-flow data prior to 1927 have been published by the U. S. Geological Survey,[8] and later data are taken from unpublished records in the Survey files.

The best rainfall records in this general region are those of the Mount Wilson and Santa Anita Ranger Stations. Both stations are operated and the records published by the U. S. Weather Bureau. As it was not anticipated that a study of denudation would be made in either area, detailed climatological and other records, such as those which were collected with great care and accuracy in the Wagonwheel Gap area, are lacking for this study.

No records of temperature were collected directly in the Southern California area during the first or second period of the study. At Mount Wilson, on the western divide of Santa Anita Creek, the average annual temperature during the two periods was 55.7° Fahr., and the daily temperature ranged from about 100° Fahr. in summer to less than 32° Fahr. in winter.

Effect of Deforestation and Denudation

Annual Run-Off.—In the following tables the period of record before deforestation and denudation is designated the "first period," and the period of record after deforestation and denudation, the "second period." The precipitation given for the Wagonwheel Gap area (Table 2) is the average of Area *A* and Area *B*; that for the Southern California area is the precipitation recorded at Mount Wilson.

In determining the effect of deforestation and denudation on these areas it was first necessary to ascertain what the run-off of Area *B* and the Fish

[8] *Water Supply Papers Nos. 481, 511, 531, 551, 571, 591. 611,* and *631,* U. S. Geological Survey.

Creek Basin would have been if conditions of vegetative cover had not been changed by deforestation or denudation. Although the two contiguous areas, *A* and *B* at Wagonwheel Gap, appeared to be alike in many respects, their run-off relations varied greatly. For example, Area *B* before deforestation had a daily run-off as low as 50% of that of Area *A* during the rising flood stages and as high as 150% during the falling stages. (See Fig. 3.) The

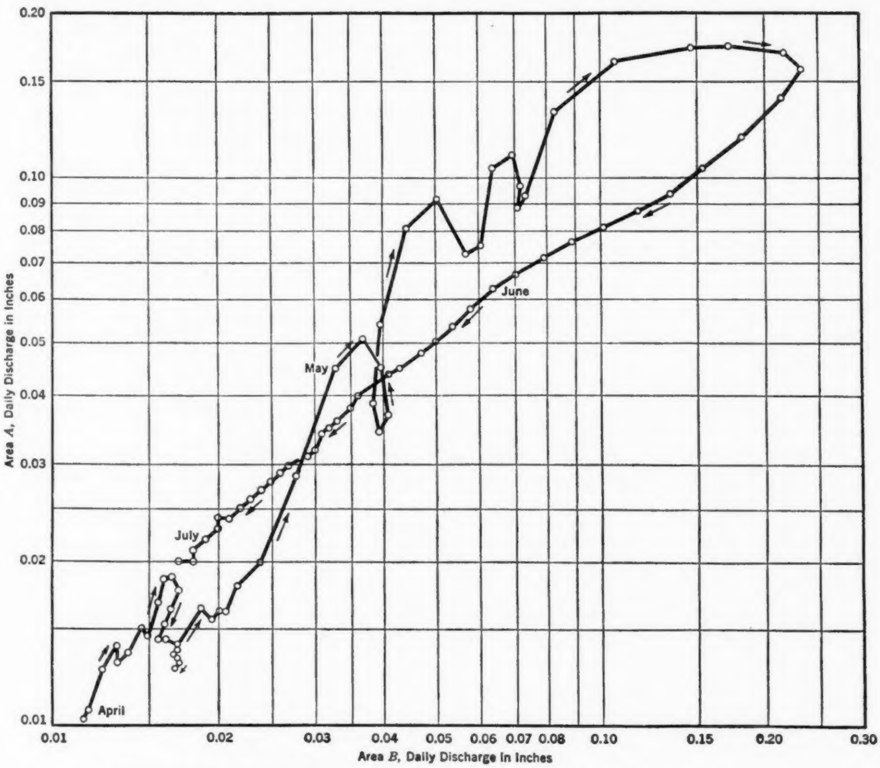

FIG. 3.—RELATION OF DAILY DISCHARGE, AREAS *A* AND *B* (WAGONWHEEL GAP) DURING RISING AND FALLING STAGES, APRIL 1 TO JULY 5, 1912

relation varied also during the remainder of the year, depending to a considerable extent on the magnitude of the daily discharge and the season. Variations in discharge relations also occurred in the Southern California area on Fish and Santa Anita Creeks.

In view of these variations, the writers believed that it was desirable to compute the probable daily discharge for the period after deforestation or denudation rather than to rely on monthly figures. This permitted a much more thorough analysis of the available data.

The normal daily discharge on Area *B* was determined by comparisons established prior to deforestation, using the following methods: Beginning in October, 1919, with the falling stage on Area *B* and continuing through July,

TABLE 2.—MONTHLY PRECIPITATION AND RUN-OFF, IN INCHES, DURING THE FIRST PERIOD

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
WAGONWHEEL GAP AREA													
1911–1912:													
Precipitation	4.54	1.15	1.50	0.35	0.40	3.06	1.54	0.34	2.17	4.18	1.74	0.43	21.40
Run-off: Area *A*	0.901	0.456	0.367	0.312	0.267	0.308	0.460	2.951	1.044	0.566	0.413	0.323	8.370
Run-off: Area *B*	0.984	0.433	0.362	0.337	0.307	0.338	0.504	2.928	0.962	0.497	0.384	0.335	8.371
1912–1913:													
Precipitation	2.56	0.44	0.70	1.05	1.09	1.48	0.82	0.52	3.10	2.11	2.40	2.60	19.16
Run-off: Area *A*	0.339	0.300	0.286	0.278	0.227	0.284	0.795	0.802	0.586	0.356	0.264	0.261	4.779
Run-off: Area *B*	0.380	0.350	0.339	0.329	0.289	0.323	0.564	1.020	0.690	0.378	0.274	0.278	5.214
1913–1914:													
Precipitation	0.97	1.85	2.37	2.20	0.66	0.74	0.94	2.29	1.63	5.06	2.18	1.38	22.25
Run-off: Area *A*	0.287	0.255	0.249	0.241	0.216	0.271	0.545	1.639	0.769	0.491	0.362	0.304	5.628
Run-off: Area *B*	0.307	0.289	0.288	0.281	0.247	0.302	0.462	1.734	0.694	0.380	0.290	0.277	5.553
1914–1915:													
Precipitation	2.22	0.02	1.20	0.90	2.40	0.35	3.74	1.54	0.50	2.25	1.86	2.98	19.92
Run-off: Area *A*	0.308	0.238	0.226	0.259	0.218	0.240	0.460	1.621	0.835	0.402	0.294	0.253	5.354
Run-off: Area *B*	0.315	0.291	0.287	0.297	0.261	0.285	0.436	1.584	0.810	0.337	0.258	0.247	5.407
1915–1916:													
Precipitation	0.36	1.98	2.72	3.48	0.49	1.60	1.99	0.34	0.09	5.21	3.14	1.52	22.92
Run-off: Area *A*	0.274	0.245	0.226	0.219	0.207	0.365	0.652	1.721	0.604	0.407	0.373	0.303	5.593
Run-off: Area *B*	0.281	0.271	0.268	0.269	0.248	0.373	0.570	1.784	0.555	0.325	0.323	0.289	5.555
1916–1917:													
Precipitation	4.37	0.20	1.48	1.89	0.74	1.58	4.98	2.27	0.13	1.94	2.08	1.17	22.83
Run-off: Area *A*	0.424	0.341	0.254	0.236	0.211	0.249	0.610	3.258	2.622	0.700	0.419	0.320	9.643
Run-off: Area *B*	0.457	0.382	0.322	0.303	0.266	0.295	0.580	2,962	3.097	0.550	0.341	0.287	9.843
1917–1918:													
Precipitation	0.19	1.07	0.27	1.29	1.92	2.08	0.72	0.14	1.04	3.84	3.20	3.12	18.88
Run-off: Area *A*	0.317	0.271	0.253	0.233	0.199	0.222	0.315	0.420	0.288	0.236	0.201	0.240	3.195
Run-off: Area *B*	0.321	0.313	0.311	0.303	0.268	0.316	0.336	0.372	0.283	0.248	0.215	0.250	3.534
1918–1919:													
Precipitation	1.05	2.52	1.88	0.06	1.80	2.58	2.10	1.20	0.89	4.37	1.04	1.53	21.14
Run-off: Area *A*	0.238	0.222	0.223	0.217	0.191	0.223	0.782	2.138	0.803	0.480	0.309	0.255	6.079
Run-off: Area *B*	0.252	0.241	0.237	0.233	0.206	0.237	0.681	2.284	0.677	0.362	0.289	0.270	5.970

TABLE 2.—(*Continued*)

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
SOUTHERN CALIFORNIA AREA													
1917–1918:													
Precipitation	0	0.45	0.05	0.64	13.39	12.85	0.37	0.31	0.07	0.40	0.39	1.90	30.82
Run-off: Santa Anita Creek	0.07	0.08	0.08	0.09	0.80	3.97	0.75	0.40	0.21	0.10	0.07	0.06	6.68
Run-off: Fish Creek	0.04	0.09	0.09	0.10	0.99	5.93	0.73	0.35	0.15	0.03	0.02	0.03	8.55
1918–1919:													
Precipitation	0.58	4.79	2.53	1.12	4.05	3.96	0.74	0.16	0	Trace	Trace	4.05	21.98
Run-off: Santa Anita Creek	0.07	0.13	0.20	0.13	0.28	0.34	0.19	0.13	0.06	0.02	0.01	0.02	1.58
Run-off: Fish Creek	0.05	0.13	0.24	0.17	0.34	0.52	0.23	0.15	0.03	0	0	0.01	1.87
1919–1920:													
Precipitation	1.37	1.72	4.36	0.84	6.95	10.12	1.70	0.08	0	0	0	0.03	27.17
Run-off: Santa Anita Creek	0.06	0.08	0.27	0.12	0.45	1.87	0.82	0.33	0.16	0.06	0.03	0.03	4.28
Run-off: Fish Creek	0.04	0.09	0.37	0.14	0.80	3.10	1.04	0.43	0.18	0.02	0	0	6.21
1920–1921:													
Precipitation	2.25	2.08	1.73	5.66	1.44	8.77	1.06	11.04	0	0.03	0	0.22	34.28
Run-off: Santa Anita Creek	0.05	0.07	0.11	0.31	0.21	1.29	0.28	1.46	0.49	0.19	0.08	0.04	4.58
Run-off: Fish Creek	0.03	0.08	0.12	0.40	0.16	1.94	0.28	1.34	0.42	0.06	0.02	0.01	4.86
1921–1922:													
Precipitation	1.12	0.11	29.40	12.86	9.90	3.76	1.91	1.20	0	0	0	0	60.26
Run-off: Santa Anita Creek	0.07	0.05	7.59	5.00	7.83	3.99	2.15	1.37	0.75	0.40	0.27	0.17	29.64
Run-off: Fish Creek	0.02	0.03	5.82	4.47	8.03	3.29	1.85	1.27	0.64	0.30	0.12	0.06	25.90
1922–1923:													
Precipitation	0.55	4.39	9.05	3.33	1.39	1.50	3.40	0	0.18	0.02	0	0.67	24.48
Run-off: Santa Anita Creek	0.17	0.37	1.35	0.48	0.54	0.38	0.42	0.27	0.18	0.12	0.06	0.04	4.38
Run-off: Fish Creek	0.09	0.31	1.64	0.53	0.64	0.38	0.47	0.17	0.08	0.02	0.02	0.02	4.37
1923–1924:													
Precipitation	0.68	0.36	1.03	1.86	0.30	9.26	3.07	0.01	0	0	0	0	16.57
Run-off: Santa Anita Creek	0.05	0.09	0.10	0.12	0.10	0.33	0.26	0.13	0.05	0.01	0.01	0.01	1.26
Run-off: Fish Creek	0.03	0.04	0.07	0.09	0.07	0.43	0.20	0.06	0.01	0	0	0	1.00

the daily discharge on Area *A* was plotted against that of Area *B*. It was found by inspection that the relation depended to a considerable extent on the quantity of the run-off. With this in mind, the records were divided into three groups: That of 1912 and 1917, which represented the years of greatest run-off; that of 1914, 1915, 1916, and 1919, the medium years; and that of 1913 and 1918, the years of low run-off. The daily relation was plotted for each of these groups on separate sheets. A curve was then drawn averaging all the points plotted. Fig. 4 is a typical curve of relation between the run-

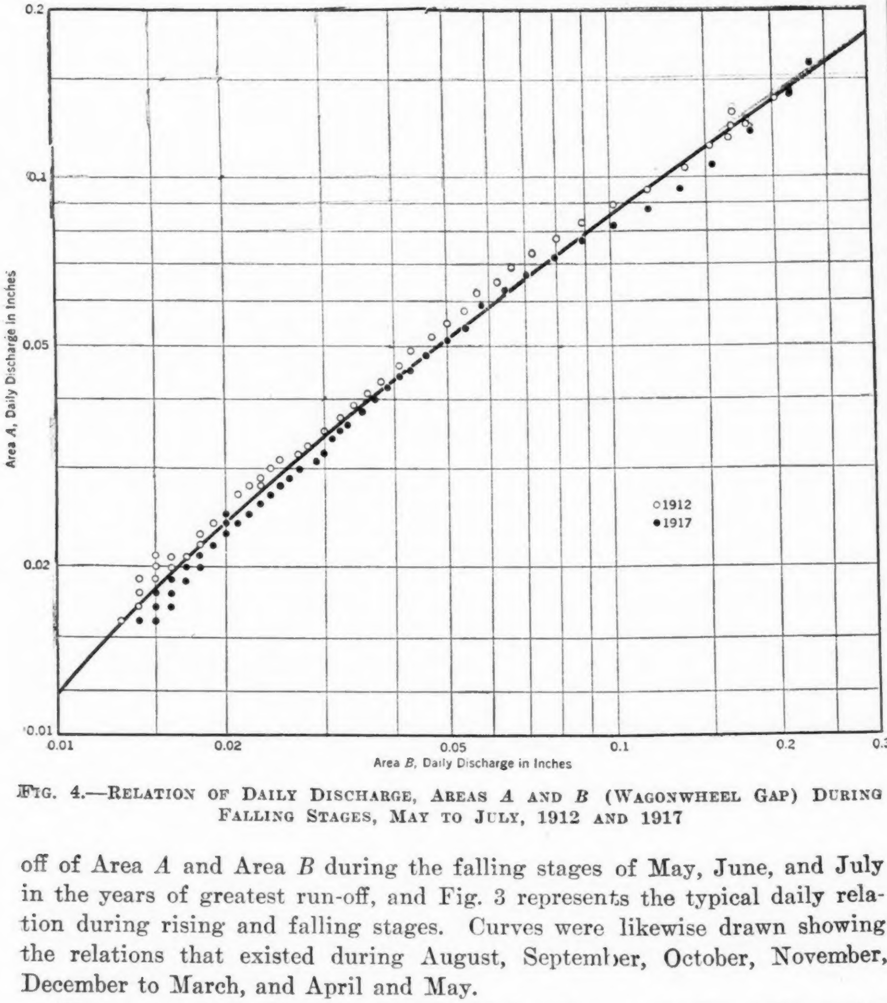

FIG. 4.—RELATION OF DAILY DISCHARGE, AREAS *A* AND *B* (WAGONWHEEL GAP) DURING FALLING STAGES, MAY TO JULY, 1912 AND 1917

off of Area *A* and Area *B* during the falling stages of May, June, and July in the years of greatest run-off, and Fig. 3 represents the typical daily relation during rising and falling stages. Curves were likewise drawn showing the relations that existed during August, September, October, November, December to March, and April and May.

The normal daily discharge in Area *B* for the period after deforestation was assumed to be the discharge obtained by applying the observed daily discharge of Area *A* to the appropriate curves, with one exception. It was noted that prior to the deforestation about three days was usually required

for the change from the rising stage curve of April and May to the falling stage curve of May to July. During this three-day transition period it was necessary to make an interpolation for Area *B* based on the record from Area *A*.

For the Southern California area, curves were drawn showing the relations, prior to the fire, for January to March, April, May, June, July, August, September, and October to December. In plotting curves the observations made during 1921-22 were not used, as in that year considerable of the precipitation occurred in the form of snow, the run-off from which represented an entirely different relation. None of the seasons following denudation on Fish Creek was comparable to 1921-22.

Table 3 shows the monthly data for the second period and, in addition, the computed normal run-off, based on the relations previously described, of Area *B* and Fish Creek as it would have been had there been no change in vegetative cover. The difference between the actual and the computed normal run-off is also shown in Table 3.

The values in the last column of Table 3 show that the maximum gain on Area *B* occurred the third year after deforestation and amounted to 2.141 in., or 32.3%, and that there was a gradual decrease for the remainder of the

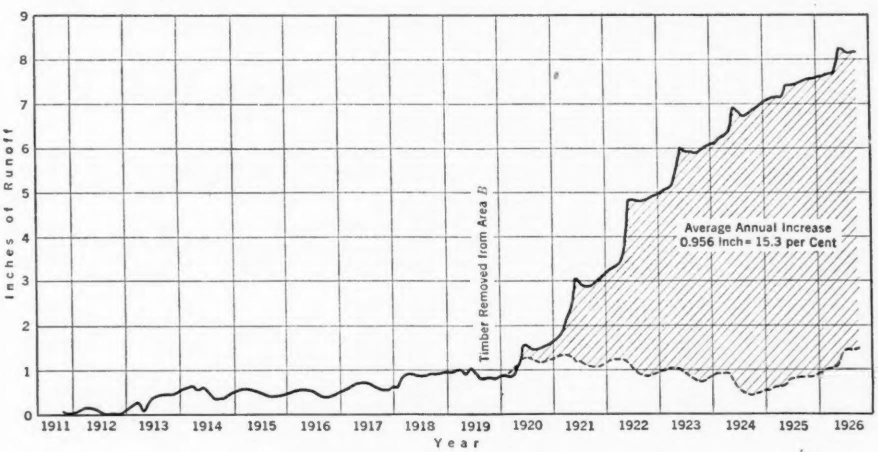

FIG. 5.—ACCUMULATED EXCESS OF RUN-OFF FOR AREA *B* OVER AREA *A*, WAGONWHEEL GAP, COLORADO

period. On Fish Creek the maximum gain occurred the second year after the fire and amounted to 3.12 in., or 26 per cent. The gain on Fish Creek the first year after the fire was 2.47 in., or 231 per cent.

Figs. 5 and 6 show the accumulated difference in run-off of Areas *A* and *B* and of Fish and Santa Anita Creeks during the first and second periods. The cross-hatched area in Fig. 5 represents an average annual increase of 0.956 in., or 15.3%, from Area *B* during the first seven years after complete deforestation. The dashed line from 1920 to 1926 represents the normal

TABLE 3.—MONTHLY PRECIPITATION AND RUN-OFF, IN INCHES, AND CHANGE IN RUN-OFF DURING THE SECOND PERIOD

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
WAGONWHEEL GAP AREA													
1919–1920: Precipitation	1.98	4.81	0.82	0.57	1.49	1.91	2.16	2.45	1.13	1.96	1.06	1.78	22.14
Run-off: Area *A*	0.276	0.254	0.250	0.237	0.218	0.241	0.347	3.583	1.292	0.503	0.357	0.305	7.865
Run-off: Area *B*:													
Actual	0.285	0.270	0.273	0.263	0.243	0.295	0.555	4.010	1.235	0.451	0.340	0.330	8.550
Normal	0.286	0.287	0.293	0.281	0.261	0.286	0.359	3.887	1.299	0.414	0.312	0.288	8.253
Increase,* in inches	−0.001	−0.017	−0.020	−0.018	−0.018	+0.009	+0.196	+0.123	−0.064	+0.037	+0.028	+0.042	+0.297
Increase, percentage	−0.3	−5.9	−6.8	−6.4	−6.9	+3.2	+54.6	+3.2	−4.9	+8.9	+9.0	+14.6	+3.6
1920–1921: Precipitation	3.42	1.88	1.52	1.36	0.60	1.08	1.58	2.51	1.75	3.28	3.05	0.55	22.59
Run-off: Area *A*	0.316	0.291	0.278	0.266	0.240	0.339	0.567	2.240	1.083	0.534	0.416	0.328	6.898
Run-off: Area *B*:													
Actual	0.364	0.360	0.351	0.336	0.318	0.703	0.737	2.927	0.977	0.493	0.397	0.364	8.328
Normal	0.326	0.327	0.314	0.307	0.277	0.367	0.491	2.130	1.157	0.443	0.353	0.306	6.798
Increase*, in inches	+0.038	+0.033	+0.037	+0.029	+0.041	+0.336	+0.246	+0.797	−0.180	+0.050	+0.044	+0.058	+1.529
Increase, percentage	+11.7	+11.0	+11.8	+9.5	+14.8	+91.5	+50.1	+37.4	−15.5	+11.3	+12.5	+18.9	+22.5
1921–1922: Precipitation	0.80	0.32	1.50	2.95	1.50	2.72	1.65	2.47	1.20	1.77	3.37	0.74	20.98
Run-off: Area *A*	0.324	0.292	0.282	0.262	0.237	0.284	0.457	2.549	1.001	0.447	0.396	0.298	6.830
Run-off: Area *B*:													
Actual	0.397	0.366	0.346	0.348	0.311	0.377	0.862	3.641	0.996	0.428	0.379	0.317	8.769
Normal	0.336	0.325	0.318	0.301	0.274	0.323	0.429	2.416	0.910	0.373	0.340	0.282	6.627
Increase*, in inches	+0.061	+0.041	+0.028	+0.047	+0.034	+0.054	+0.433	+1.225	+0.086	+0.055	+0.039	+0.035	+2.141
Increase, percentage	+18.1	+12.6	+8.8	+15.6	+12.4	+16.7	+10.1	+50.8	+9.4	+14.7	+11.5	+12.4	+32.3
1922–1923: Precipitation	0.64	4.81	1.16	0.79	0.94	1.68	2.08	0.66	1.00	2.52	4.04	3.75	24.08
Run-off: Area *A*	0.316	0.300	0.294	0.277	0.245	0.272	0.446	2.064	0.738	0.427	0.366	0.345	6.091
Run-off: Area *B*:													
Actual	0.350	0.349	0.348	0.325	0.289	0.334	0.698	2.680	0.697	0.382	0.351	0.363	7.168
Normal	0.328	0.332	0.328	0.315	0.282	0.311	0.419	1.991	0.673	0.357	0.318	0.317	5.971
Increase*, in inches	+0.022	+0.017	+0.020	+0.010	+0.007	+0.023	+0.279	+0.689	+0.024	+0.025	+0.033	+0.046	+1.195
Increase, percentage	+6.7	+5.1	+6.1	+2.9	+2.5	+7.4	+66.6	+34.6	+35.7	+7.0	+10.4	+14.5	+20.0
1923–1924: Precipitation	2.60	1.67	1.34	0.60	1.06	3.15	1.44	0.72	0.17	1.87	1.16	1.16	16.86
Run-off: Area *A*	0.388	0.348	0.314	0.282	0.257	0.278	0.920	2.518	0.789	0.428	0.305	0.276	7.104
Run-off: Area *B*:													
Actual	0.453	0.411	0.358	0.340	0.316	0.353	1.006	3.093	0.646	0.391	0.316	0.332	8.016
Normal	0.411	0.401	0.346	0.319	0.293	0.317	0.754	2.396	0.675	0.359	0.276	0.264	6.811
Increase*, in inches	+0.042	+0.010	+0.012	+0.021	+0.023	+0.036	+0.252	+0.697	−0.029	+0.032	+0.40	+0.068	+1.204
Increase, percentage	+10.2	+2.5	+3.5	+6.6	+7.9	+11.4	+33.4	+29.2	−4.3	+8.9	+14.5	+25.8	+17.7
1924–1925: Precipitation	1.48	1.06	2.16	0.16	0.58	2.97	1.46	1.18	2.16	3.82	3.70	1.46	22.15
Run-off: Area *A*	0.316	0.289	0.288	0.274	0.244	0.294	0.599	0.696	0.440	0.308	0.267	0.254	4.269
Run-off: Area *B*:													
Actual	0.382	0.351	0.345	0.324	0.294	0.355	0.588	0.934	0.447	0.338	0.308	0.283	4.948
Normal	0.327	0.322	0.322	0.313	0.280	0.330	0.590	0.830	0.488	0.314	0.275	0.248	4.639
Increase*, in inches	+0.055	+0.029	+0.023	+0.011	+0.014	+0.025	−0.002	+0.104	−0.041	+0.024	+0.033	+0.035	+0.310
Increase, percentage	+16.8	+9.0	+7.1	+3.5	+5.0	+7.5	−0.4	+12.5	−8.4	+7.6	+12.0	+14.1	+6.69

*Negative sign denotes decrease.

TABLE 3.—(*Continued*)

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
WAGONWHEEL GAP AREA — (*Continued*).													
1925–1926: Precipitation	2.30	1.66	0.12	0.68	0.38	2.17	1.47	1.91	0.95	2.55	2.28	1.70	18.16
Run-off: Area *A*	0.279	0.252	0.247	0.231	0.208	0.236	0.444	1.068	0.588	0.352	0.260	0.212	4.379
Run-off: Area *B*:													
Actual	0.310	0.286	0.274	0.258	0.236	0.291	0.442	1.612	0.540	0.320	0.254	0.228	5.050
Normal	0.290	0.286	0.292	0.277	0.250	0.282	0.416	1.327	0.765	0.367	0.271	0.218	5.032
Increase*, in inches	+0.020	0.000	−0.018	−0.019	−0.014	+0.009	+0.026	+0.285	−0.216	−0.047	−0.017	+0.010	+0.019
Increase, percentage	+6.9	......	−6.2	−6.9	−5.6	+0.3	+6.3	+21.5	-28.5	-12.8	−6.3	+4.6	+0.04
SOUTHERN CALIFORNIA AREA													
1924–1925: Precipitation:													
Mount Wilson	1.06	2.74	2.89	0.33	2.77	3.60	6.16	0.97	1.39	0	0	0	21.91
Santa Anita Ranger Station	1.80	3.05	2.86	0.32	1.89	2.62	6.54	1.23	0.84	0	0	0	21.15
Run-off:													
Santa Anita Creek	0.03	0.07	0.11	0.09	0.10	0.11	0.45	0.14	0.10	0.02	0.01	0.01	1.24
Fish Creek:													
Actual	0.01	0.10	0.26	0.10	0.14	0.50	1.92	0.24	0.16	0.06	0.03	0.02	3.54
Normal	0.01	0.05	0.12	0.07	0.07	0.09	0.51	0.10	0.05	0	0	0	1.07
Increase*, in inches	0	+0.05	+0.14	+0.03	+0.07	+0.41	+1.41	+0.14	+0.11	+0.06	†0.03	+0.02	+2.47
Increase, percentage	∞	+100	+117	+43	+100	+456	+276	+140	+220	∞	∞	∞	+231
1925–1926: Precipitation:													
Mount Wilson	1.65	2.72	2.95	2.66	7.78	0.64	17.55	0.31	0	0	0	0	36.26
Santa Anita Ranger Station	2.88	1.78	3.87	2.46	8.11	0.52	18.92	1.06	0	0	0	0	39.60
Run-off:													
Santa Anita Creek	0.02	0.05	0.09	0.09	0.66	0.13	5.70	0.62	0.23	0.10	0.05	0.03	7.77
Fish Creek:													
Actual	0.05	0.06	0.24	0.16	1.96	0.39	10.85	0.63	0.29	0.13	0.09	0.07	14.92
Normal	0.01	0.03	0.08	0.08	0.83	0.12	9.76	0.66	0.19	0.03	0.01	0	11.80
Increases*, in inches	+0.04	+0.03	+0.16	+0.08	+1.13	+0.27	+1.09	−0.03	+0.10	+0.10	+0.08	+0.07	+3.12
Increase, percentage	+400	+100	+200	+100	+136	+225	+11.2	−4.5	+52.6	+334	+800	∞	+26
1926–1927: Precipitation:													
Mount Wilson	0.10	5.53	2.97	1.84	17.07	4.16	1.56	0.14	0	0	0	0	33.37
Santa Anita Ranger Station	0.11	6.00	3.17	2.72	19.77	†	†	0.80	0.15	0	0	0	....
Run-off:													
Santa Anita Creek	0.03	0.21	0.28	0.26	4.43	1.90	1.02	0.50	0.31	0.16	0.08	0.04	9.24
Fish Creek:													
Actual	0.09	0.44	0.34	0.46	8.36	2.34	1.11	0.63	0.41	0.21	0.14	0.12	14.65
Normal	0.01	0.23	0.31	0.33	7.62	2.39	1.25	0.54	0.27	0.07	0.03	0.02	13.07
Increases†, in inches	+0.08	+0.21	+0.03	+0.13	+0.74	−0.05	−0.14	+0.09	+0.14	+0.14	+0.11	+0.10	+1.58
Increase, percentage	+800	+91	+9.7	+39.4	+9.7	−2.1	−11.2	+16.7	+51.9	+200	+367	+500	+12.1

*Negative sign denotes decrease.
†Incomplete.

TABLE 3.—(*Continued*)

Precipitation and run-off	October	November	December	January	February	March	April	May	June	July	August	September	The year
SOUTHERN CALIFORNIA AREA — (*Continued*).													
1927–1928: Precipitation:													
Mount Wilson	3.13	1.18	5.85	0.30	4.35	3.02	0.88	1.07	0	0	0	0	19.78
Santa Anita Ranger Station	2.94	1.34	†	†	3.37	3.32	1.42	1.10	0	0	0	0	....
Run-off:													
Santa Anita Creek	0.05	0.14	0.27	0.20	0.37	0.31	0.21	0.14	0.06	0.02	0.01	0.01	1.79
Fish Creek:													
Actual	0.13	0.27	0.48	0.28	0.47	0.31	0.24	0.17	0.07	0.03	0.01	0.01	2.47
Normal	0.02	0.15	0.32	0.23	0.41	0.33	0.20	0.11	0.02	0	0	0	1.79
Increase†, in inches	+0.11	+0.12	+0.16	+0.05	+0.06	−0.02	+0.04	+0.06	+0.05	+0.03	+0.01	+0.01	+0.68
Increase, percentage	+550	+80	+50	+21.7	+14.6	−6.1	+20	+54.5	+250	∞	∞	∞	+38
1928–1929: Precipitation:													
Mount Wilson	0.95	2.10	3.30	3.42	3.59	2.94	3.84	0	0.31	0	0	0.23	20.68
Santa Anita Ranger Station	0.80	2.19	4.87	2.75	2.97	3.81	4.92	0	0.36	0	0	0.47	23.14
Run-off:													
Santa Anita Creek	0.02	0.05	0.17	0.17	0.30	0.43	0.74	0.18	0.08	0.02	0.01	0.01	2.17
Fish Creek:													
Actual	0.05	0.08	0.32	0.30	0.51	0.73	0.74	0.18	0.08	0.01	0	0.01	3.01
Normal	0	0.04	0.19	0.19	0.32	0.50	0.89	0.15	0.04	0	0	0	2.32
Increase*, in inches	+0.05	+0.04	+0.13	+0.11	+0.19	+0.23	−0.15	+0.03	+0.04	+0.01	0	+0.01	+0.69
Increase, percentage	∞	+100	+68.5	+57.9	+59.4	+46.0	−16.8	+20	+100	∞	0	∞	+29.7
1929–1930: Precipitation:													
Mount Wilson	0.07	0	0	8.39	1.22	8.02	1.29	4.01	0	0	0	0	23.00
Santa Anita Ranger Station	0.16	0	0	9.47	1.25	6.90	0.93	3.97	0	0	0	0	22.68
Run-off:													
Santa Anita Creek	0.01	0.02	0.03	0.33	0.14	0.78	0.26	0.47	0.15	0.04	0.02	0.01	2.26
Fish Creek:													
Actual	0.02	0.03	0.04	0.65	0.13	1.24	0.25	0.53	0.15	0.03	0.01	0.01	3.09
Normal	0	0	0.01	0.43	0.13	0.91	0.26	0.50	0.10	0.01	0	0	2.35
Increase*, in inches	+0.02	+0.03	+0.03	+0.22	−0	+0.33	−0.01	+0.03	+0.05	+0.02	+0.01	+0.01	+0.74
Increase, percentage	∞	∞	+300	+51.2	−0	+36.3	−3.8	+6.0	+50	+200	∞	∞	+31.5

* Negative sign denotes decrease.
† Incomplete.

excess. In Fig. 6 an average annual increase of 1.55 in., or 28.7%, is indicated from Fish Creek during the first six years after complete denudation by the forest fire. On this diagram the dashed line from 1925 to 1930 represents the normal excess.

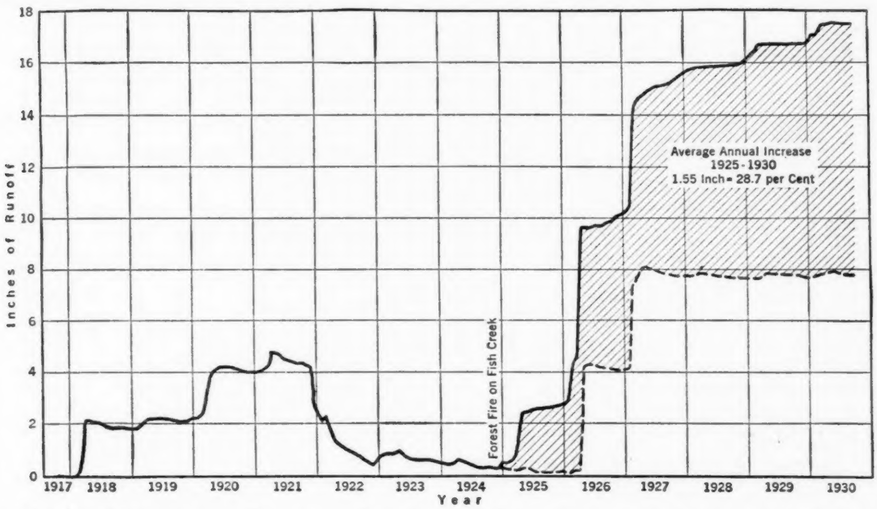

FIG. 6.—ACCUMULATED EXCESS OF RUN-OFF FOR FISH CREEK OVER SANTA ANITA CREEK IN SOUTHERN CALIFORNIA

Distribution of Increase in Annual Run-Off.—The opinion most commonly accepted is that the entire increase of run-off that follows the removal of forest cover occurs during flood periods, and that during low-water periods the run-off is decreased. It was apparent, therefore, that the increase during flood periods and also during the remainder of the year should be studied.

The floods occurring in the two areas studied are very different in type, those in the Wagonwheel Gap area being caused by melting snow as a result of high temperatures and occasional warm light rains, and those in Southern California almost entirely from rain. Stream flow in the Wagonwheel Gap area is on the increase from early April to a peak about May 20, followed by a gradual decline through the remainder of May, all of June, and the first part of July.

Table 4 shows the dates of disappearance of snow from Areas *A* and *B* during two years before and two years after deforestation; the average date of snow disappearance; and the dates of the maximum daily discharge. "Date of snow disappearance" is defined by the authors of *Weather Bureau Review, Supplement No. 30,* as "the first regular observation data on which a trace or no snow was recorded, provided that at no subsequent date a depth of 3 in. or more was recorded." Between fifteen and twenty snow scales were located in each area.

These dates are presented to show the lag between the disappearance of the snow and the peak discharge in the stream. These data and the reasonable assumption that there was no sheet run-off after the snow had gone are

the basis for the statement made subsequently under the heading "Maximum Daily Discharge," that "the gain is due to subsurface flow as practically all the snow had melted prior to the date of maximum peak."

TABLE 4.—RELATION BETWEEN DISAPPEARANCE OF SNOW AND MAXIMUM DAILY DISCHARGE IN WAGONWHEEL GAP AREA

Year	Date Snow Disappeared		Date of Maximum Daily Discharge		Number of Days Between	
	Area *A*	Area *B*	Area *A*	Area *B*	Area *A*	Area *B*
FIRST PERIOD						
1913 *	April 19	April 22	May 1	May 12	12	20
1917 *	May 25	May 29	June 4	June 6	10	7
Eight years' average †	May 3	May 5	May 14	May 20	11	15
SECOND PERIOD						
1920 *	May 10	May 5	May 23	May 24	13	9
1925 *	April 23	April 20	May 11	May 15	18	19
Seven years' average ‡	May 8	May 6	May 15	May 17	7	11

* Average dates of melting are published in *Weather Bureau Review, Supplement No. 30*, for only these 4 years.
† Average for the 8 years covered by the first period.
‡ Average for the 7 years covered by the second period.

Only once during the fifteen years of record in the Wagonwheel Gap area was there a flood not attributable mainly to melted snow, and this occurred during October, 1911, prior to deforestation. On the other hand, the floods in the Southern California area may occur at any time after rains, although the most usual time is between December and May. A flood in the Southern California area comes to a peak within a few hours after the rain and seldom lasts more than three days.

In this study the term, "flood period," is roughly defined as a period of major surface run-off as distinguished from the period during which the streams are fed mainly, if not entirely, from sub-surface sources. Owing to the nature of the run-off in the California area the flood periods there are well defined and usually extend over a period of a few days. The run-off during these periods is usually greater than 4 sec-ft. per sq. mile. In the Wagonwheel Gap area the flood period is difficult of determination, but that area has been assumed to be in flood when the daily run-off exceeds 0.05 in. over the drainage basin, or 1.35 sec-ft. per sq. mile, a rate a little less than one-half the average maximum daily run-off. On the average, the period of flood run-off thus determined embraces about 25 days, generally from May 10 to June 5, or somewhat less than 10% of the time.

Table 5 shows, for both areas, the distribution of the difference between the run-off in the flood periods as defined and that for the remainder of the year. In the Wagonwheel Gap area the maximum increase occurred during the third year after deforestation. There was a decrease during the flood period in the first and last years, and the principal increases occurred during the second, third, fourth, and fifth years. The increase during the non-flood period lessened annually from the third year to the last, as would be expected.

In the Southern California area the maximum increase occurred during the second year after the fire. The increase during the flood period occurred mainly in the first three years and was practically negligible in the last three years. The increase during the non-flood period was annually less from the second year to the last.

TABLE 5.—DISTRIBUTION OF INCREASE OR DECREASE, IN INCHES, IN RUN-OFF DURING THE SECOND PERIOD

Year ending September 30	Total increase	Increase or decrease during flood periods	Increase during remainder of year	Year ending September 30	Total increase	Increase or decrease during flood periods	Increase during remainder of year
WAGONWHEEL GAP AREA				SOUTHERN CALIFORNIA AREA			
1920	0.297	−0.039	0.336	1925	2.47	+1.51	0.96
1921	1.529	+0.698	0.831	1926	3.12	+1.61	1.51
1922	2.141	+1.212	0.929	1927	1.58	+0.85	0.73
1923	1.195	+0.600	0.595	1928	0.68	+0.08	0.60
1924	1.204	+0.760	0.444	1929	0.69	+0.15	0.54
1925	0.310	+ 000	0.310	1930	0.74	+0.23	0.51
1926	0.019	−0.008	0.027	......	......	......	......

The data of Table 5 show conclusively that the increase in run-off after the deforestation and fire was not confined entirely to flood periods.

Maximum Daily Discharge and Date of Occurrence.—Table 6 shows that the maximum daily discharge, in second-feet per square mile, under natural

TABLE 6.—MAXIMUM DAILY DISCHARGE, IN SECOND-FEET PER SQUARE MILE

First Period						Second Period					
Year	Area A		Area B		Ratio, $\frac{B}{A}$	Year	Area A		Area B		Ratio, $\frac{B}{A}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
WAGONWHEEL GAP AREA											
1912	May 20	4.73	May 22	6.28	1.33	1920	May 23	6.10	May 24	8.64	1.42
1913	May 1	0.97	May 12	1.08	1.11	1921	May 13	2.16	May 15	3.95	1.83
1914	May 11	1.95	May 16	2.31	1.19	1922	May 21	3.22	May 22	5.98	1.86
1915	May 19	2.16	May 20	2.17	1.01	1923	May 20	2.36	May 21	4.28	1.82
1916	May 11	2.61	May 14	3.03	1.16	1924	May 11	2.96	May 15	5.04	1.70
1917	June 4	4.40	June 6	5.40	1.23	1925	May 11	0.74	May 11	1.08	1.46
1918	May 6	0.45	May 21	0.35	0.78	1926	May 8	1.12	May 8	1.90	1.70
1919	May 6	2.57	May 17	3.03	1.18	......	......	......	......	......	......
Average	......	2.48	......	2.95	1.19	Average	......	2.67	......	4.41	1.65

SOUTHERN CALIFORNIA AREA

Year	Santa Anita Creek		Fish Creek		Ratio, $\frac{F}{SA}$	Year	Santa Anita Creek		Fish Creek		Ratio $\frac{F}{SA}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
1918	Mar. 11	14.7	Mar. 11	29.7	2.04	1925	Mar. 6	0.10	Mar. 6	4.77	47.7
1919	Feb. 11	1.62	Feb. 11	1.54	0.95		Mar. 29	0.14	Mar. 29	4.92	35.2
1920	Mar. 2	6.8	Mar. 2	10.8	1.59		April 4	2.38	April 4	20.60	8.65
	Mar. 22	5.7	Mar. 22	12.8	2.25		April 22	0.30	April 22	4.62	15.4
1921	Mar. 13	10.4	Mar. 13	18.5	1.78	1926	Feb. 13	4.38	Feb. 13	13.7	3.13
1922	Feb. 9	33.3	Feb. 9	44.6	1.34		April 7	29.8	April 7	63.0	·2.11
1922	Dec. 13	8.57	Dec. 13	9.85	1.15	1927	Feb. 16	41.0	Feb. 16	74.2	1.81
1924	Mar. 26	1.52	Mar. 26	2.16	1.42	1927	Dec. 10	0.40	Dec. 10	1.62	4.05
						1929	Mar. 10	2.67	Mar. 10	6.31	2.36
Average	......	8.35	......	12.98	1.55	1930	Jan. 15	1.90	Jan. 15	6.46	3.40
							Mar. 15	2.38	Mar. 15	6.00	2.52

conditions from the Southern California area is more than five times greater than the maximum daily discharge from the Wagonwheel Gap area. The average ratio of the discharge of Area *B* to that of Area *A* under natural conditions was 1.19, and under deforested conditions, 1.65, an increase of 39 per cent. The average ratio of the discharge of Fish Creek to that of Santa Anita Creek under natural conditions was 1.55. During the first two years after the fire the maximum ratio was as high as 47.7. For the first four storms occurring in the first year after the fire the average increase was about 1 700%, based on a comparison of ratios. By the second year after the fire the ratios had dropped to practically normal.

In the Wagonwheel Gap area the effect of the removal of the trees was to advance the date of the maximum daily run-off of Area *B* about three days. In the Southern California area, the fire caused no appreciable change in the occurence of maximum daily discharge.

Fig. 7 shows the actual and computed normal run-off, in second-feet per square mile, of Area *B* and Fish Creek during the typical high-water period occurring three years after deforestation and denudation (April, May, and June, 1922, on Area *B*, and February, March, and April, 1927, on Fish Creek). The shaded area represents the increase or decrease of surface run-off. For clearness an insert is shown with an exaggerated time scale for a few days during the flood peak in the Fish Creek area (Fig. 7(*c*)). It will be observed that practically all the increase on both areas occurred on the rising stage. During the falling stages decreases were common.

Maximum Peak Run-Off.—In the foregoing remarks no reference has been made to peak discharges. In the Wagonwheel Gap area maximum peak discharge results from melting snow, and the ratio between maximum peak and maximum daily discharge is slightly more than 1:1. In a semi-arid country, such as the Fish Creek Basin, the maximum discharge depends almost entirely on the rainfall. Immediately after the fire the bare canyon walls were much more conducive to run-off than the plant-covered surface that

TABLE 7.—Maximum Peak Discharge and Mean Daily Discharge, in Second-Feet per Square Mile, in the Fish Creek Area, Southern California

Year	Date	Peak discharge	Daily discharge	Ratio, peak to daily discharge	Year	Date	Peak discharge	Daily discharge	Ratio, peak to daily discharge
First Period					Second Period				
1917-18...	March 10	50.8	14.7	3.5	1924-25..	Apr 4	335	20.6	16.2
1918-19...	February 11	3.2	1.5	2.1	1925-26..	April 7	292	63.0	4.6
1919-20...	March 2	39.2	10.8	3.6	1926-27..	February 16	145	74.2	2.0
1920-21...	March 13	44.0	18.5	2.4	1927-28..	February 4	14.9	4.6	3.2
1921-22...	February 9	77.7	44.6	1.7	1928-29..	March 10	10.9	6.3	1.7
1922-23...	December 12	28.6	9.8	2.9	1929-30..	January 15	11.1	6.5	1.7
1923-24...	March 26	8.9	2.2	4.0	....	....	.....	.....	....
Average..	....	36.0	14.6	2.5	....	....	.....	.....	....

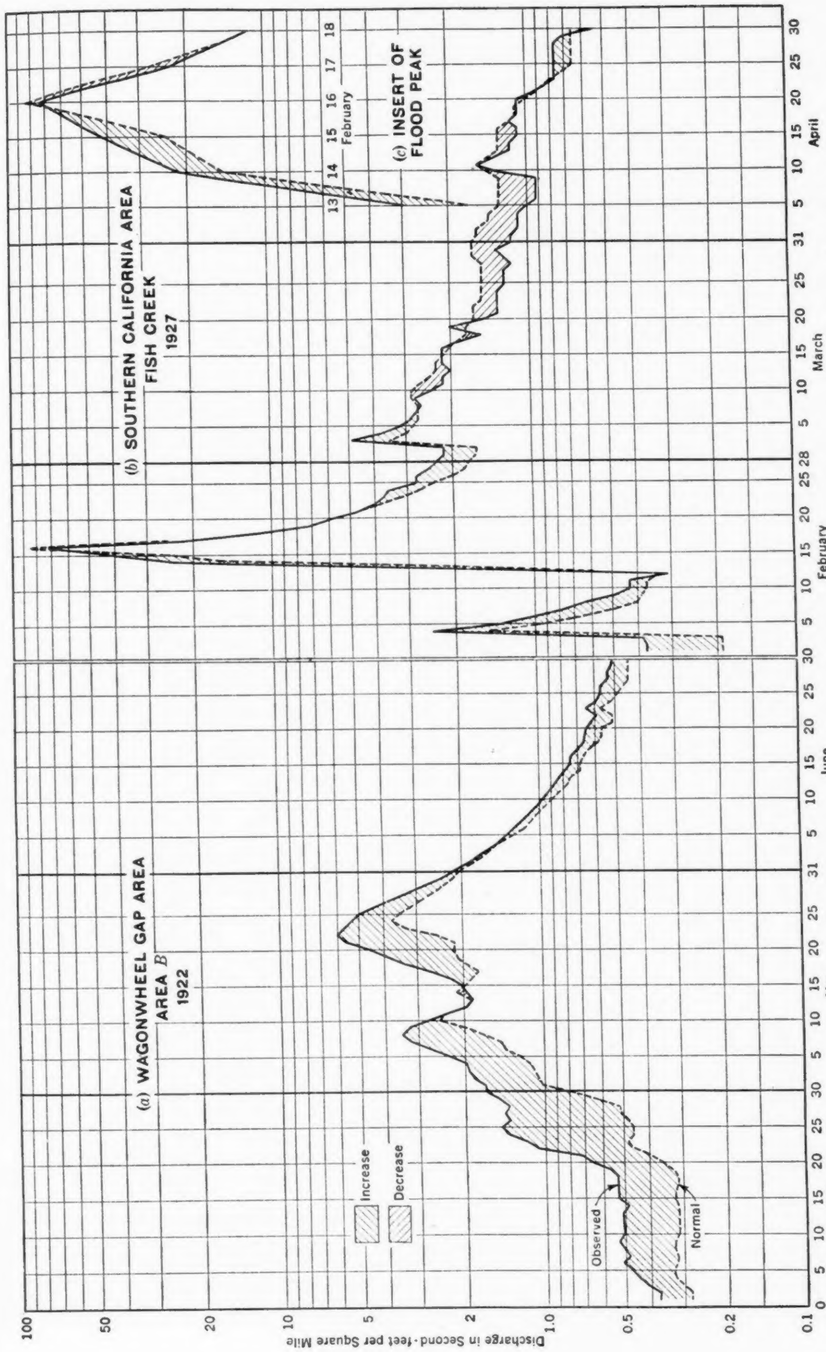

FIG. 7.—OBSERVED AND NORMAL RUN-OFF OF AREA *B* AND FISH CREEK DURING THE STORM PERIOD FOR THE THIRD YEAR AFTER CHANGE

existed prior to the fire. The greatest effect was felt during the first year after denudation. This is well expressed in Table 7, which shows the maximum peak discharges for each year, together with the mean daily discharge for the same day.

During the first period the average peak discharge was 2.5 times the mean daily discharge. In the second period, the first year (1924-25) showed a ratio considerably greater than would have been expected under normal conditions. In the second year, the ratio of the peak to the daily discharge was greatly reduced, but still was larger than any of the ratios of the first period.

Summer Run-Off.—Table 8 shows the run-off for July to October for both areas for both periods. In the Wagonwheel Gap area the ratio of the run-off of Area *B* to that of Area *A* for the first period was 0.93 and for the

TABLE 8.—SUMMER RUN-OFF (JULY TO OCTOBER), IN INCHES

First Period				Second Period			
Year	Area *A*	Area *B*	Ratio, $\frac{B}{A}$	Year	Area *A*	Area *B*	Ratio, $\frac{B}{A}$
WAGONWHEEL GAP AREA							
1912	1.641	1.596	0.97	1920	1.481	1.485	1.00
1913	1.168	1.237	1.06	1921	1.602	1.651	1.03
1914	1.465	1.262	0.86	1922	1.457	1.474	1.01
1915	1.223	1.123	0.92	1923	1.526	1.549	1.02
1916	1.507	1.394	0.90	1924	1.325	1.421	1.07
1917	1.756	1.499	0.85	1925	1.108	1.239	1.12
1918	0.915	0.965	1.05	1926	*0.824	*0.802	0.97
1919	1.320	1.206	0.91		....	....	....
Average	1.375	1.285	0.93	Average	1.333	1.375	1.03

SOUTHERN CALIFORNIA AREA

Year	Santa Anita Creek	Fish Creek	Ratio, $\frac{F}{SA}$	Year	Santa Anita Creek	Fish Creek	Ratio, $\frac{F}{SA}$
1918	0.30	0.13	0.43	1925	0.06	0.16	2.67
1919	0.11	0.05	0.45	1926	0.21	0.38	1.81
1920	0.17	0.05	0.29	1927	0.33	0.60	1.82
1921	0.38	0.11	0.29	1928	0.06	0.10	1.67
1922	1.01	0.57	0.56	1929	0.05	0.04	0.80
1923	0.27	0.09	0.33	1930	*0.07	*0.05	0.71
1924	0.06	0.01	0.17		....	....	....
Average	0.38	0.17	0.45	Average	0.13	0.22	1.69

* October record not available.

second period, 1.03. In the Southern California area the ratio of the run-off of Fish Creek to that of Santa Anita Creek for the first period was 0.45 and for the second period, 1.69.

Table 9 shows the increase in run-off for July to October in the second period for both areas. The average increase in Area *B* during the seven years after deforestation was 12%, and the average increase in the Fish Creek area for the six years after the fire was 475 per cent.

TABLE 9.—INCREASE IN SUMMER RUN-OFF, JULY TO OCTOBER, IN INCHES AND PERCENTAGE, DURING THE SECOND PERIOD

WAGONWHEEL GAP AREA				SOUTHERN CALIFORNIA AREA			
Year	Normal discharge	Increase		Year	Normal discharge	Increase	
		Inches	Percentage			Inches	Percentage
1920...........	1.340	0.145	11	1925.........	0.01	0.15	1 500
1921...........	1.438	0.213	15	1926.........	0.05	0.33	660
1922...........	1.323	0.151	11	1927.........	0.14	0.46	304
1923...........	1.403	0.146	10	1928.........	0	0.10	∞
1924...........	1.226	0.195	16	1929.........	0	0.04	∞
1925...........	1.127	0.112	10	1930.........	*0.01	*0.04	*400
1926...........	*0.856	*0.074	*9		....	....	....
Average.......	1.244	0.148	12	Average......	0.04	+0.19	475

* October not available.

Fig. 8 shows the actual and the computed normal run-off during August and September, the period of low summer run-off, for the third year after

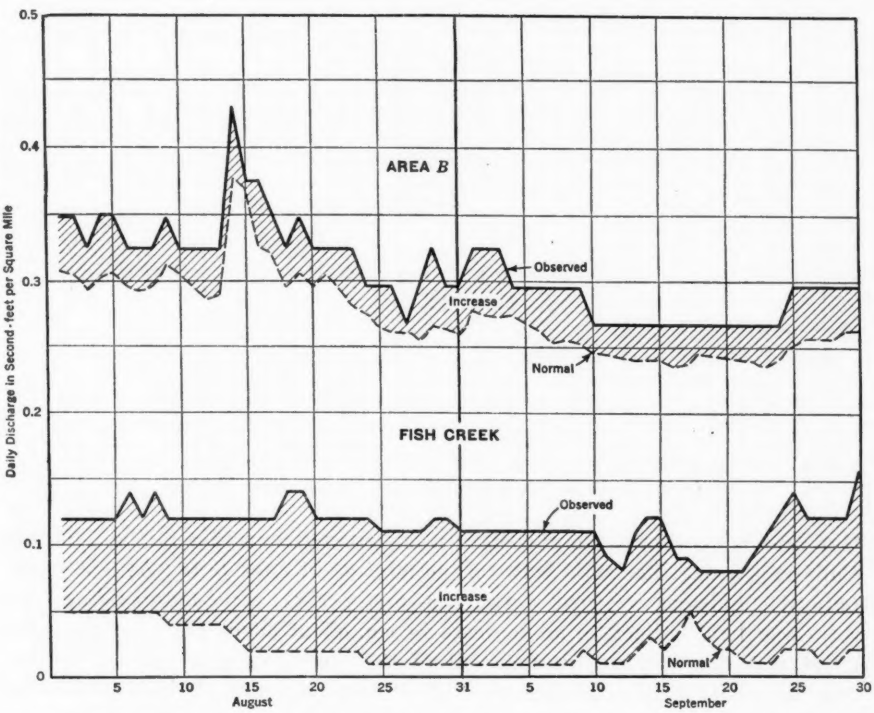

FIG. 8.—OBSERVED AND NORMAL SUMMER RUN-OFF OF AREA *B* AND FISH CREEK FOR THE THIRD YEAR AFTER THE CHANGE

the change (1922 for Area *B* and 1927 on Fish Creek). The cross-hatched area represents the typical increase in run-off during the summer.

Fig. 9 shows graphically the increase in summer for both areas for the entire period. This diagram shows the accumulated differences in run-off for the two areas for the first period and the accumulated differences between the observed run-off and the normal run-off of both areas for the second period.

Minimum Daily Discharge and Date of Occurrence.—Table 10 shows the minimum daily discharge during the summer for both areas for both periods. Under normal conditions, the minimum rate of discharge in the Wagonwheel

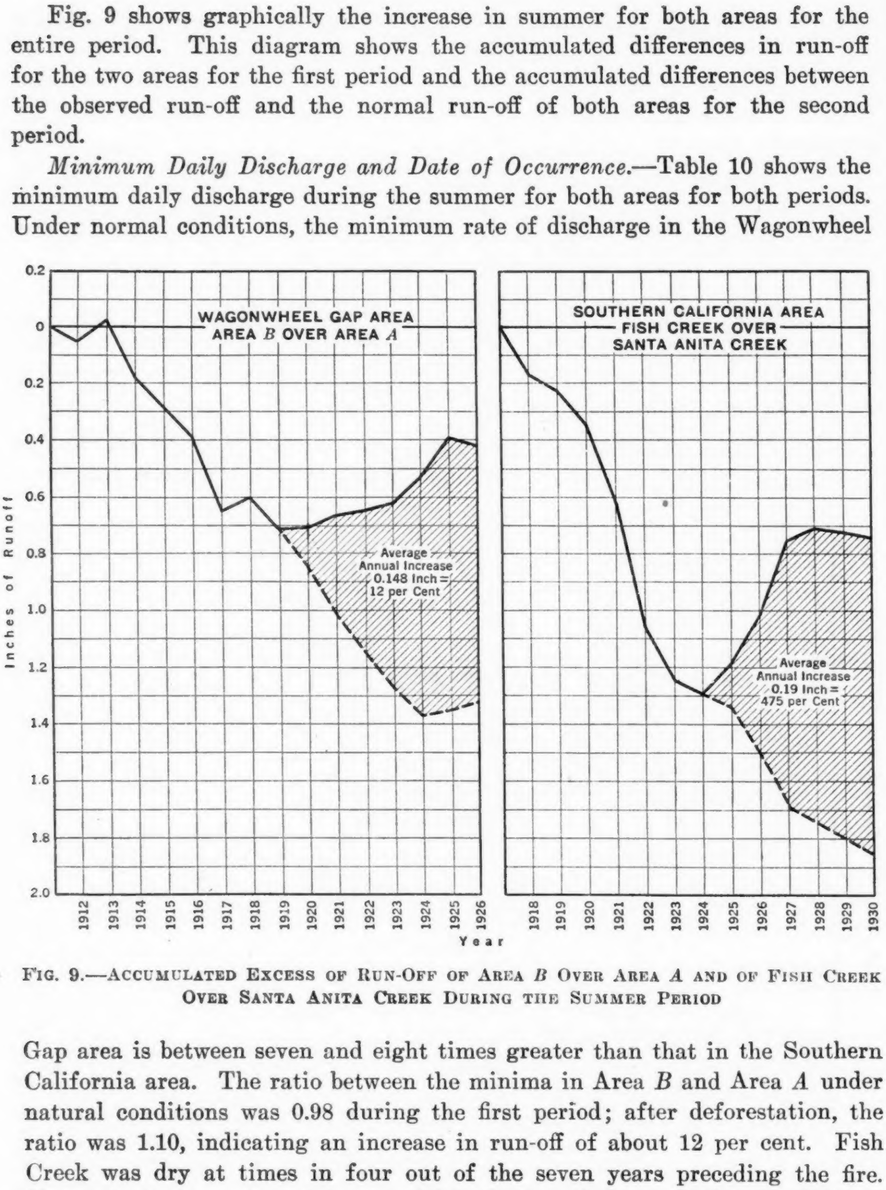

FIG. 9.—ACCUMULATED EXCESS OF RUN-OFF OF AREA *B* OVER AREA *A* AND OF FISH CREEK OVER SANTA ANITA CREEK DURING THE SUMMER PERIOD

Gap area is between seven and eight times greater than that in the Southern California area. The ratio between the minima in Area *B* and Area *A* under natural conditions was 0.98 during the first period; after deforestation, the ratio was 1.10, indicating an increase in run-off of about 12 per cent. Fish Creek was dry at times in four out of the seven years preceding the fire. The ratio between the minima on Fish Creek and Santa Anita Creek based on the seven years was 0.32.

After the removal of the forest cover in Area *B,* the date of occurrence of the summer minimum was delayed about five days. After the fire in the Fish Creek area the date of occurrence of the summer minimum was delayed more than a month.

Table 11 shows the minimum daily discharge during the winter for the Wagonwheel Gap area. The average ratio of the minimum in Area *B* to that in Area *A* during the first period was 1.30, or 1.24 if 1915, when the winter minimum for Area *A* was abnormal, is excluded. The ratio during the second period was 1.27 indicating little if any change in winter minima. Inasmuch as the date of occurrence and to a large extent the magnitude of winter minima are a direct result of low temperatures no material change was to be expected. In the Southern California area winter temperatures are not sufficiently low to cause a period of corresponding low run-off.

TABLE 10.—Minimum Daily Discharge During Summer, in Second-Feet per Square Mile

First Period						Second Period					
Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$	Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
Wagonwheel Gap Area											
1912	September 16	0.283	September 7	0.279	0.99	1920	August 30	0.253	August 31	0.265	1.05
1913	August 31	0.205	August 8	0.221	1.08	1921	September 14	0.276	September 14	0.312	1.13
1914	September 8	0.247	September 7	0.224	0.91	1922	September 17	0.247	September 14	0.265	1.08
1915	September 12	0.183	September 12	0.183	1.00	1923	September 11	0.256	September 11	0.298	1.16
1916	August 28	0.259	August 28	0.237	0.92	1924	August 31	0.230	August 22	0.250	1.09
1917	September 6	0.268	September 4	0.236	0.88	1925	August 17	0.205	August 17	0.232	1.13
1918	August 25	0.159	August 1	0.171	1.08	1926	September 10	0.173	September 1	0.179	1.03
1919	September 4	0.207	September 4	0.225	1.09	....	....	.....	....	.....	....
Average	....	0.226	....	0.222	0.98	Average	....	0.234	....	0.257	1.10
Southern California Area											
Year	Santa Anita Creek		Fish Creek		Ratio, $\frac{B}{A}$	Year	Santa Anita Creek		Fish Creek		Ratio, $\frac{B}{A}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
1918	September 11	0.03	July 11	0.02	0.67	1925	July 19	0.01	August 18	0.02	2.00
1919	July 31	0.01	June 26	0	∞	1926	August 25	0.02	September 20	0.05	2.50
1920	August 9	0.02	July 29	0	∞	1927	August 25	0.04	September 12	0.08	2.00
1921	September 6	0.02	August 27	0	∞	1928	July 23	0.01	August 20	0	∞
1922	September 25	0.11	September 25	0.03	0.27	1929	August 18	0	August 4	0	∞
1923	September 2	0.02	July 11	0.02	1.00	1930	August 22	0.01	July 27	0	∞
1924	July 2	0.01	June 15	0	∞	....	....	.....	....	.....	....
Average	....	0.031	....	0.010	0.32	Average	....	0.015	....	0.025	1.67

Erosion: Wagonwheel Gap Area.—In connection with the studies of run-off, quantitative measurements were made, by means of settling basins, of the silt carried by the streams of the two areas near Wagonwheel Gap. During the first eight years the average annual quantity of silt deposited in the collection basin was 691.5 lb. for Area *A* and 568.5 lb. for Area *B*, or 3.15 and 2.85 lb., respectively, per acre. During the seven years after deforestation the average quantities collected were 477 lb. from Area *A* and 3 340 lb. from Area *B*, or 2.15 and 16.7 lb., respectively, per acre, showing about an eight-

fold increase in the ratio of Area *B*. The higher value represents only a few shovelfuls per acre per year, and, as Messrs. Bates and Henry state,[9]

"It is only fair to the present discussion to point out that even this large quantity does not represent erosion in the commonly accepted sense of a destructive process."

As to the visible effect, they say:

"The erosion from the slopes was practically invisible except for one small gully formed from a skid trail, and even the erosion from this appeared to be largely deposited upon a leveled road, without reaching the stream channel."

TABLE 11.—Minimum Daily Discharge During Winter, in Second-Feet per Square Mile, in the Wagonwheel Gap Area

Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$	Year	Area *A*		Area *B*		Ratio, $\frac{B}{A}$
	Date	Discharge	Date	Discharge			Date	Discharge	Date	Discharge	
First Period						Second Period					
1912	Feb. 28	0.234	Feb. 28	0.279	1.19	1920	Mar. 8	0.195	Feb. 15	0.222	1.14
1913	Mar. 1	0.210	Mar. 16	0.266	1.27	1921	Feb. 10	0.220	Feb. 10	0.278	1.26
1914	Feb. 14	0.203	Feb. 16	0.233	1.15	1922	Jan. 27	0.170	Jan. 23	0.289	1.70
1915	Dec. 16	0.109	Dec. 17	0.216	1.98	1923	Mar. 18	0.225	Mar. 15	0.273	1.21
1916	Feb. 6	0.182	Feb. 1	0.227	1.25	1924	Feb. 5	0.221	Feb. 6	0.283	1.28
1917	Dec. 11	0.188	Dec. 15	0.266	1.41	1925	Feb. 23	0.231	Feb. 26	0.273	1.18
1918	Feb. 28	0.178	Feb. 3	0.248	1.39	1926	Feb. 27	0.191	Feb. 6	0.214	1.12
1919	Feb. 17	0.180	Feb. 1	0.195	1.08	......	....	.....	....	.....	....
Average: Including 1915......	....	0.185	....	0.241	1.30	Average.	....	0.207	....	0.262	1.27
Omitting 1915......	....	0.197	....	0.245	1.24						

Erosion: Southern California Area.—On Fish Creek the rapid run-off during the first year after the fire (1924-25) caused considerable erosion to the side walls of the canyon. Beginning December 16, 1924, a series of silt samples were taken in connection with the current meter measurements made at irregular intervals by the U. S. Geological Survey at each of the gauging stations in the burned area. Most of these samples were collected at times when the discharge in the streams was at storm stage. For this reason the figures obtained are considerably above the average for the entire year.

The first rains after the fire washed the ash and loose material from the side walls of the canyon into the stream, to such an extent that by December 16, 1924 (three months after the fire), the entire stream bed had been buried. From a series of pools and rocky rapids, it had become a channel of more or less uniform cross-section with a fairly uniform gradient, sufficient to cause a considerable increase in the velocity. Three current meter measurements made on April 4, 1925, at discharges of 105 to 74 sec-ft., showed mean velocities of 10.5 to 9 ft. per sec. Under normal conditions this discharge would not have had a velocity of more than 2 ft. per sec. As late as June 4, 1925, when the discharge was 3.1 sec-ft., the mean velocity was 4.8 ft. per sec. To this uniform gradient of the stream bed with the increased velocity is partly due the rapid run-off and the unusually large peak storm discharge.

[9] *Monthly Weather Review, Supplement 30.*

The silt samples were collected in quart jars. As a rule the mouth of the jar was held half way between the bottom and the surface of the stream. Owing to the shallowness of the stream and the greatly increased velocity, part of the material collected in the samples consisted of the heavier particles which, ordinarily, would form the rolling load, but were then temporarily in suspension.

After the samples had been allowed to settle for three years the results shown in Table 12 were obtained. As a rule, it was possible to separate the ash from the sand, because the sand settled first with the ash on top of it. In most of the samples the division between the sand and ash was well marked. The material called sand in Table 12 consisted mainly of small irregular particles washed off the canyon walls. Included with the ash was some unburned organic matter which usually floated on the surface.

TABLE 12.—Silt in Samples from Southern California Area

Location	Percentage by Volume			Percentage by Weight		
	Sand	Ash	Total	Sand	Ash	Total
Rogers Creek:						
December 16, 1924, 11:30 A. M.	25	42	67	25	14	39
December 16, 1924, 3:30 P. M.	18	38	56	18	13	31
March 6, 1925	25	25	50	32	6	38
March 29, 1925, 10:25 A. M.	28	16	44	28	6	34
April 4, 1925, 9:55 A. M.	24	24	48	25	10	35
April 20, 1925, 9:20 P. M.	10	12	22	......	......	9.8
April 7, 1926, 5:30 P. M.	......	......	5.5	......	......	3.8
Fish Creek:						
December 16, 1924, 12:20 P. M.	......	......	20	......	......	5.9
March 31, 1925	......	......	20	......	......	14.3
Sawpit Creek:						
December 16, 1924, 2:00 P. M.	10	40	50	12	14	26
April 4, 1925	30	36	66	30	10	40

Under normal conditions the erosion in each of the three areas represented in Table 12 is negligible. Prior to the fire there was practically no movement of silt on Fish Creek except during the storm run-off, and by the third or fourth day after a storm the water was practically clear. In the first year after the fire the large deposit of silt from the burned-over area caused considerable damage to orchards, railroads, and highways adjacent to the mountains. In the second year the quantity of silt moved was much smaller and caused little if any damage. By 1930, the silt had been reduced to almost normal, and the appearance of the stream bed was much the same as before the fire.

Conclusions

The conclusions to be drawn from this paper may be discussed under seven heads, as follows:

1.—Total Run-Off.—Forests did not "conserve the water supply," because after their removal there was an increase in average annual yield amounting to 15% in a mountain area in Colorado and 29% in a Southern California coastal mountain area.

2.—Distribution of Increase in Run-Off.—Contrary to the widely quoted opinion the increase in run-off is not confined wholly to flood periods. In

both the Wagonwheel Gap area and in the Southern California area, 52% of the increase occurred during the non-flood period. The flow during the non-flood period is derived from sub-surface storage. The increase during non-flood periods results from either (*a*) increased sub-surface flow and storage; or (*b*) decreased transpiration; or (*c*) a combination of both. More of the precipitation may enter storage after removal of vegetative covering for the following reasons: (1) Less interception by trees, undergrowth, tree litter, and humus; and (2) faster melting of snow with corresponding decrease in evaporation. The gradual lessening of the increase during the non-flood period in both areas after the second or third year may be a reflection of gradual increase in plant growth and corresponding increase in transpiration.

3.—Maximum Daily Discharge.—In the Wagonwheel Gap area there was an average increase of 46% in maximum daily discharge after deforestation. This gain is due to increased sub-surface flow, as practically all the snow had melted several days prior to the date of maximum peak. During flood periods following the snow-melting the discharge in Area *B* uniformly reached a peak three days or more after that in Area *A*, but after the storm of October 5, 1911, the peak in both areas occurred at the same time. This was the only so-called flood that resulted from rainfall and indicates that surface run-off from both areas reached the gauging station at practically the same time. In the Southern California area the four storms occurring during the first year after the fire resulted in an increase of 1 700% in the maximum daily discharge. The peak discharge, which was ordinarily 2.5 times the maximum daily discharge prior to the fire, increased to 16.2 times on April 4, 1925, the maximum peak for the period of record. The floods in the Southern California area usually result from rainfall and represent direct surface run-off.

The removal of vegetative covering clearly increases normal flood heights. Beginning in the second year after the fire on Fish Creek, the flood peak discharges were practically the same as those which occurred before the fire (see Table 7), indicating that the new growth, small as it was, exercised practically the same effect as the original cover in reducing flood crests.

The gradual increase in vegetative covering on the Wagonwheel Gap Area *B* after deforestation had little effect on the increase in flood run-off. (See Table 6.) This was to be expected in that most of the flood run-off in this area reaches the stream through underground channels.

The earlier melting of the snow in the Wagonwheel Gap area resulted in an advance of three days in the flood peak. Where rain passes directly into streams without entering the ground the flood peak for any small element of drainage area occurs so soon after the storm that removal of vegetative cover has little effect on the time element. In the Southern California area the peak was advanced only a few hours.

4.—Summer Run-Off.—It is almost universally believed that forests or vegetative covering will increase summer run-off and shorten the low-water period through the exercise of storage functions. This belief is an outstanding fallacy in so far as these two widely different areas are concerned.

The summer flow (July to October) in the Wagonwheel Gap area showed an average annual increase of 12% during the seven years after deforestation, and the Southern California area, an average annual increase of 475% during the six years after the fire. The absolute average increase of run-off, in inches, for both areas was practically the same. (See Table 7.) The high percentage of increase shown for Fish Creek results from the extremely small summer run-off of that stream under normal conditions. This increase is probably a result of increase in sub-surface storage and decrease in transpiration.

5.—*Minimum Daily Discharge and Date of Occurrence.*—Coincident with the increase in summer run-off there was an increase in the average summer minimum and the period of low-water run-off was considerably shortened. In the Wagonwheel Gap area the average minimum was increased about 12% and the time of occurrence delayed about 5 days. (See Table 10.) In the Southern California area the average minimum was increased more than 400% and the time of occurrence was delayed about 30 days.

6.—*Winter Minimum.*—Deforestation made no appreciable change in the low flows which occurred during the winter in the Wagonwheel Gap area.

7.—*Erosion.*—Erosion results from surface flow. In the Wagonwheel Gap area there was practically no evidence of erosion after deforestation, and this was to be expected because there was little direct surface run-off either before or after deforestation. In the Southern California area complete denudation increased erosion as a direct result of the increased surface run-off. So far as the surface features of the Fish Creek Basin were concerned the erosion did not destroy any present use. However, deposition of eroded material and ash carried by the streams the first year after the fire was materially injurious to agricultural lands and transportation rights of way below the canyon. As it is clearly shown in Tables 6 and 7 that the growth of new vegetative covering by the second year after the fire was nearly as effective in reducing normal flood run-off as the original cover, it may well be argued that if the Fish Creek area had been deforested by cutting of the vegetative cover, as in the Wagonwheel Gap area, without the destruction of the roots and tree litter, the erosion would have been practically the same as with the original vegetative covering.

Deductions

If the facts as found are generally applicable to other similar areas which in their natural state support forest growth, then the hydraulic engineer and the water user must weigh such protective influence as forest cover may have in lowering normal flood crests and in preventing or retarding erosion against the detriment resulting from deceased yield and reduced low-water flow during the critical growing period. If the small growth that springs up immediately after deforestation or denudation exercises practically the same effect as forests in reducing normal flood crests and in preventing erosion—without the detrimental effect which forest cover is shown to have on annual flow and flow during the summer low-water periods—then in basins where shortages in water supply are becoming critical or where abnormal expendi-

tures have been made to augment water supplies, the maintenance of forests or reforestation for the "conservation of water supply" may have an effect exactly opposite to that desired.

The writers are lovers of forests and have a keen appreciation of the value of water in the economic development of the United States. The value of the forests as a source of wood for building material and other industrial purposes and as playgrounds is alone sufficient to warrant their preservation and maintenance. Scientifically determined facts apparently do not warrant their development solely for the conservation of water supply. In regions where such supply is a controlling economic factor, careful study is needed to determine whether the value of increased water supply and better-sustained minimum flow which are shown to obtain without forests, does not outweigh the benefits of lowered normal flood flows and decreased erosion produced by forests, especially if these benefits can be obtained by shrubs or other small growth without the loss of water occasioned by forest growth.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FULTON STREET, EAST RIVER TUNNELS, NEW YORK, N. Y.

Discussion

BY MESSRS. OLE SINGSTAD, JOHN F. O'ROURKE, AND H. H. HATCH

OLE SINGSTAD,[7] M. AM. SOC. C. E. (by letter)[7a]—This paper is a concise and well prepared account of an important and interesting construction job, well executed. Two outstanding features of this work are most impressive: (1) The rapid progress attained in shield driving, which has established a new record for subaqueous tunneling in this kind of ground; and (2) the driving of the shields through Fulton Street below and very close to the foundations of buildings of considerable height, without underpinning the buildings and without causing serious settlement or damage to them. This result, of course, could not have been attained without the effective measures taken, as described in the paper, to insure the instant and complete backfilling of the void created outside the cast-iron lining in advancing the shield.

It is also interesting to note certain improvements over previous practice in the details of the plant, such as the use of automatic or push-button elevators in hoisting materials up and down the shafts (an arrangement which no doubt was prompted by the scarcity of labor and the high wages paid at the time); and the use of a simple mechanical device for saturating the hemp grommets with red lead. This device not only effected a saving in labor, but unquestionably resulted in a more thorough saturation of the grommets, thereby improving their water-proofing properties.

The erection of the shield on a cradle equipped with rollers also is a novel feature, resulting in greater ease in erecting and equipping the shield and a saving in the time required for this work.

The rapid progress made on the work and the excellent results obtained are due largely to the fact that the contractor, as well as the City, had organizations of construction men and engineers in charge, with extensive experience in tunnel work, and that the job was equipped with an adequate and up-to-date plant.

NOTE.—The paper by Miles I. Killmer, M. Am. Soc. C. E., was presented at the meeting of Construction Division, New York, N. Y., January 16, 1930, and published in December, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1932, by Messrs. H. J. King, S. M. Swaab, and Jacob Feld.

[7] Chf. Cons. Engr. on Tunnels, The Port of New York Authority, New York, N. Y.

[7a] Received by the Secretary February 17, 1932.

JOHN F. O'ROURKE,[8] M. AM. SOC. C. E. (by letter).[8a]—In addition to this paper there are only two papers[9] in the publications of the Society on the subject of under-river shield tunnels and their approaches. The papers are all well written, fully illustrated, and treat the subject from two different points of view; namely, that of the owner's engineers who supervised the work in the field and that of the engineer who was in actual charge of the work for the contractor. It would be well if descriptions of the construction record of the numerous other shield tunnels under the Hudson River and the East River had been published by the Society so that the growth of the art of shield tunneling in New York City could be followed from one to another, including the views of the engineers and contractors. At any rate the facts concerning them are generally known and short references to some of them may be of value in showing the successive improvements by which shield tunneling has reached the high state of development disclosed in Mr. Killmer's extremely lucid and important paper.

The first under-river shield tunnels in New York City were the two under the Hudson River (The Hudson and Manhattan Tunnel) from Morton Street, Manhattan, to Jersey City, N. J. (begun in 1874). These were started as brick tunnels constructed without shields by the Haskins System, which were later changed to shield tunnels lined with cast iron. The brick tunnels were driven through the clay river bed with 34 lb. of compressed air by excavating the neat size of the brickwork and lining the roof and sides of the excavation, as it progressed, with light steel rings, 30 in. wide, in 3 and 6-ft. long segments, that broke joints with those of adjoining rings. This steel lining was braced radially with timbers from a pilot tube at the center, 6 ft. in diameter, which was driven a few feet in advance of the heading. The 24-in. brick lining was built as each 15-ft. length of excavation was completed. The muck from the next 15-ft. length was filled into the last length of brickwork to a little below the springing line (upon which the construction tracks were extended), after which the remainder of the excavation for the length was completed and hauled to the shaft in muck cars drawn by mules. It is significant that the material from the heading was sufficiently firm to support the construction track and give a footing for the mules without planking, and confirms what follows on that subject.

A paper[10] on this tunnel by the late Arthur Spielmann and Charles B. Brush, Members, Am. Soc. C. E., relates to the work at the beginning and before the method of construction described herein had been fully developed. However, the paper has many facts concerning the steel-plate rings. The material of the river bed through which the tunnel passed, is described as so stiff under the air pressure that it could be cut in benches, with steps

[8] Cons. Engr., New York, N. Y.

[8a] Received by the Secretary April 20, 1932.

[9] "The New York Tunnel Extension of the Pennsylvania Railroad—The North River Tunnels," by B. H. M. Hewett and W. L. Brown, Members, Am. Soc. C. E.; and "The East River Tunnels," by J. H. Brace, Francis Mason, and S. H. Woodard, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXVIII (1910), pp. 152 and 419, respectively.

[10] "The Hudson River Tunnel," *Transactions*, Am. Soc. C. E., Vol IX (1880), p. 259.

rising from one bench to the other. Another paper[11] by the late William Sooy Smith, M. Am. Soc. C. E., gives further information regarding the river bed and describes the Haskins tunneling methods in detail. This comparative solidity of the so-called silt, which is in reality a sedimentary deposit of clay in horizontal layers, was subsequently confirmed by Messrs. Aims and Fitzgerald, Resident Engineer and Superintendent, respectively, in numerous personal interviews which contained a wealth of corroborative detail, and also by the writer's own observations in front of a shield where these tunnels were being driven through part rock and clay near the New York side of the river. He also had opportunities to observe the clay of the river bed at the Pennsylvania Railroad tunnels; at Haverstraw, N. Y., where the clay for brick-making is taken in great quantities from behind coffer-dams in the river, the clay banks being like those on shore; and also at the Poughkeepsie Bridge during the sinking of the pier foundations. In all these cases the clay was similar to that previously described, through which the Hudson and Manhattan Railroad Company's tunnels were driven so that the entire river bed from New York to Poughkeepsie may be said to consist of a fairly firm and dependable clay with a shallow covering of mud where the clay has been disturbed. Attention is called to these facts as showing the reliable nature of the bed of the river where undisturbed for the firm support of tunnels and other structures with properly designed foundations, because there is a widespread belief that the river bed is soft and affords only insecure support. The direct opposite is true, particularly as to tunnels, which weigh less than the material they replace, so that the clay beneath them has less load on it than before their construction.

It is a fact that at both the Pennsylvania Railroad tunnels and the Holland Tunnel the clay surrounding the tunnels directly behind the shields became soft and slushy for a few feet out from the tunnel lining and that some settlement as well as flotation of the tunnels occurred. This was occasioned by the ground not closing in on the lining at the tail of the shield, but maintaining itself for some time as a dirt tunnel of the diameter of the shield. The weight of the overlying ground and water caused arch stresses in the surrounding parts greatly in excess of the ordinary compression stresses when directly supported from below, and squeezed the firm clay composing the face of the dirt tunnel into a watery mud.

This same clay when forced during shoves into the tunnel through openings in the bulkheads, by hydraulic pressures exceeding 100 lb. per sq. in. on the heading in front of the shield, required a foot on the spade to cut into it during removal. This shows how firm the clay surrounding the tunnels naturally is and that if immediate support were provided for the tunnel and surrounding ground as the shield was shoved (as can easily be done), the destructive arch stresses would be prevented and the tunnel would be as free of either settlement or flotation as were the brick tunnels in the same kind of ground, where immediate support by the steel lining plates preserved the firmness of the clay unchanged.

[11] "The Hudson River Tunnel," *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 314.

The Hudson and Manhattan Railroad Company's tunnels are also of interest because in them cast-iron shield tunneling was introduced in New York City through the advice of the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E. Certain London bankers retained Sir Benjamin to report on these tunnels when they were considering a loan to the Tunnel Company. He learned that on two occasions when the air pressure in the northerly brick tunnel, extending from Jersey City to the middle of the Hudson River, was reduced from 34 lb. to 20 lb., because of trouble with the steam boilers, the heading began to move into the tunnel. This was stopped both times by rapid and effective bracing. The possibility that, on some future occasion, when the air pressure might be lowered or lost, the bracing might fail and the tunnel be filled with silt, seemed to Sir Benjamin too great a risk to ignore, and he advised the bankers to make the use of shields a condition of the loan, which was done. This involved abandonment of the brick lining and the substitution of cast-iron lining, which was the only kind known at that time that could resist the thrust of shield jacks. The existing brick tunnels were entirely satisfactory in his judgment and subsequent events have justified this opinion. The northern tunnel (the western half of which was built of brick at that time and the eastern half subsequently built of cast iron) has been in constant and satisfactory use to its full capacity for twenty-three years (since 1907), and the brick and cast-iron parts are both in equally fine condition.

The next shield tunnels undertaken (1903) were for the Rapid Transit Commission of New York from South Ferry, Manhattan, to Joralemon Street, Brooklyn, with approaches in Battery Park, Manhattan, and Joralemon Street, Brooklyn. These tunnels were of cast iron, as are all subsequent river tunnels and approaches in New York at the present. Shield tunneling being still quite undeveloped, there were difficulties and stoppages of work, but after some delays and much discussion, the troubles were corrected, the tunnels were finished, and have been giving satisfactory service for more than twenty years.

The tunnel construction along Joralemon Street in Brooklyn caused immediate settlement of the street over the tunnels as the shields advanced, a little later followed by settlement of the stoops and areas, outward from the buildings. Later, the fronts of the buildings began to settle and crack, the settlement of the ground under them progressing slowly to the rear for several years, until the angle of rupture of the ground was finally reached when the settlements stopped. By this time some of the buildings had moved out several inches into the street, spur shores against the fronts were frequently used until the settlements stopped, and a number were so badly cracked that the front walls had to be rebuilt. After years of litigation, the Court of Appeals affirmed the verdicts for damage to the buildings and disallowance of the much larger claims for contingent damages, such as losses of rentals and interference with business. The opinion stated in support of this that the work had been done without negligence, that the grout used to minimize settlement, while it was the best known method, had not entirely succeeded in doing so, and that the damages were incidental to the character of the work.

FIG. 19.—NORTH SIDE OF OLD SLIP, SHOWING WHERE GROUT WAS USED OUTSIDE OF LINING ALLOWING SETTLEMENT OF STREET OVER TUNNEL.

FIG. 20.—NORTH SIDE OF OLD SLIP, SHOWING WHERE GRAVEL WAS USED OUTSIDE OF LINING PREVENTING SETTLEMENT OF STREET OVER TUNNEL.

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This means that the best known method of preventing settlement must be used if consequential damages are to be avoided.

The shield tunnels undertaken after the Pennsylvania Railroad tunnels, were across the East River, two from Whitehall Street, Manhattan, to Montague Street, Brooklyn, and two from Old Slip, on the New York side, to Clark Street, in Brooklyn. It was on these contracts that the writer introduced gravel packing to fill the space left around the tunnel lining by the shields. The contracts specified grout for this purpose, paid for per barrel of cement used. After gravel packing had been tried for a short time and was found to prevent visible settlement of streets or buildings the contracts were modified to substitute it for the grout used immediately behind the shields.

The gravel packing was first used in the river headings from the shaft at Old Slip and Front Street on the New York side. Figs. 19 and 20 show Old Slip on January 29, 1916, about two months after the tunnels were driven beneath. Gravel packing was supposed to be used from the beginning, but the street settled about 6 in. for nearly 75 ft. When the writer went into the tunnel it was found that, although the blowing apparatus and pea gravel were provided, the men did not understand the method and had confined their activities to grouting. The writer instructed them in getting the apparatus in working order and in blowing the gravel into the space around the tunnel at the rear of the shield, so that in a few hours the tunnel and ground at the rear of the shield were firmly supported for the length of a ring. When the next shove was made the operation was repeated and from that time on gravel packing the space left by the shield was started with the shove and continued until the entire space was packed, in the order of top, sides, and bottom, and there was no further settlement.

In Fig. 19 a white arrow shows where the settlement was stopped at the tail of the shield. The line marks the end of a piece of curb where the shield had not moved, of which the other end had settled 5 in. From this point the curb did not settle perceptibly, as the gravel packing was regularly used with a success that was warmly appreciated by the tunnel men, many of whom had worked on the Joralemon Street tunnels and believed shield tunneling and settlements to be inseparable.

Fig. 20 is a view from the corner of South Street looking backward to the shaft at Front Street, Manhattan. The sidewalk and roadway in this picture as far as the white arrow, remained without settlement. The line of the curb shows this, and any unevenness in the sidewalk and pavement was there before the tunnel was driven. Neither view shows settlement in the adjoining building as its pile foundations were carried on solid rock.

The same successful avoidance of visible settlement followed the construction of the Brooklyn approaches on Clark and Montague Streets where the gravel packing was faithfully performed. There was equal success in preventing settlements in driving the tunnels on Fulton and Willoughby Streets, Brooklyn.

Grout was used in all thesee tunnels from one to three days after the gravel packing had been placed. The gravel packing was so dense that grout

could not be forced through it and a hole had to be punched through the gravel at each grout hole before the grout hose nipple was attached. In excavations subsequently made around the tunnels for inter-track stations, the grout was usually found in large lumps in the ground where it emerged through the gravel as there were no holes or cracks into which it could flow. It appeared that when the grout came in contact with the surrounding ground that the water was filtered from it, the sand and cement forming a lump that grew in size until the resistance of the ground to further expansion of the lump caused a refusal of flow from the grout mixer under a pressure usually about 100 lb. per sq. in. These lumps greatly compacted the ground surrounding them.

The collection of grout into lumps around the tunnel, which was invariably the case so far as the excavations disclosed at the Clark and Montague Street inter-track stations, goes far to explain the settlements already described when the Joralemon Street tunnels were driven. In that case grout alone was used, and it must have made pockets for itself in the ground outside the grout holes and formed lumps like those described. The extent of lateral compression of the ground by these lumps was limited, so that the greater part of the empty space around the tunnels was left unsupported. In any event were the character of the ground to permit the entire empty space to be filled with grout, it would be too soft for at least an hour to support the overlying ground which usually starts to settle at once.

Mr. Killmer states that the success of the entire tunneling procedure was linked to the successful placing of the gravel packing, and that this was so well done that only a few minor cracks were caused in driving the tunnels. This latter fact is very important regarding that contract as none of the high buildings was underpinned, while all buildings of seven stories or more were underpinned before the Montague and Clark Street tunnels were driven past them, because the security afforded by gravel packing was not then known. An examination to-day of underpinned buildings and those adjoining them on Montague Street would show the joints between them to be undisturbed although in some cases the walls of the small buildings extend only 10 ft. below the streets while the underpinning of the high buildings near the river extends 100 ft. below the street. The fact is that gravel packing prevents all but a very slight settling of the ground as a whole along the tunnels, which levels show to be uniform whether the buildings be high or low, so that underpinning is unnecessary, as was proved on Fulton Street.

It was a great pleasure to read Mr. Killmer's compact and comprehensive paper. Its modest tone does not obscure the fact that the organization that did that work is entitled to the greatest credit for skill, energy, and care, which, when all the difficulties are taken into account, make the construction of these tunnels a classic.

H. H. HATCH,[12] M. AM. SOC. C. E. (by letter).[12a]—The author has covered his subject-matter thoroughly; that is, contractors' problems and plants. In connection with his instructive description, it would have been interesting

[12] Engr. in Chg., Cobble Mountain Reservoir, Springfield Water-Works, Westfield, Mass.

[12a] Received by the Secretary May 14, 1932.

to know also about the engineering problems, quantities, and unit costs of this work.

It always pays to make the necessary precautions for the safety of life and the expedient progress of the work, especially in constructions of more or less risky nature. Sometimes a contractor tries to "get by" with plant on hand and without making the necessary preparations for emergency. At times, he succeeds, and often he has "hard luck" which results in loss of life, time, and money. Mr. Killmer, however, seems to have taken the necessary precaution for any emergency with respect to his equipment and personnel.

It is interesting to note that the many tall buildings near-by the tunnel could be held in place without settlement and without any support other than a maximum air pressure of 48 lb. per sq. in. Disregarding the disturbance of the original ground in driving the shields, it means that a force of about 3.5 tons per sq. ft. was sufficient to balance the reaction of the tall buildings on their foundations.

Evidently the results of back-fill with pea gravel were satisfactory, as "no building settled or suffered injuries more than minor cracks." It occurs to the writer, however, that the more uniform the material sizes the higher is the percentage of voids; and that if there is any variation in the particle sizes of the gravel, or if pea gravel is mixed with sand, before being shot into the openings outside the shield, the back-fill would become more compact due to a smaller percentage of voids. It is desirable to know the maximum and minimum pressures used in shooting the pea gravel back of the shields.

More information on the grouting would have been welcome. Practical experience on various tunnel and foundation grouting has proved that better work can be obtained with neat cement than with an admixture of sand. The thinner the grout the farther is its penetration. Often, with sand-cement grout it appears that the sand, being heavier, will stop short during the discharge of the grout and let the fluid travel past it. This process gradually will form sand pockets near the discharge of the grout pipe and eventually will prevent the passage of more grout.

In a rock tunnel with concrete lining, it is possible to compute the minimum quantity of grout necessary to fill the voids back of the concrete. To this amount, of course, should be added the estimate of grout necessary to seal up the rock seams, if any. It has been found that the grout will shrink and that the excess water, especially in thin mixtures, will occupy space, provided the pressure does not drive it away. Often it is this excess water that will find weak spots in the concrete lining and seep through, thus guiding the engineer in locating at least the outlet of water from voids for additional grouting. It is questionable whether, in shield tunneling, the quantity of grout can be estimated in advance. It would be instructive to know just what methods were used to make positively sure that the grouting was complete. Were there any investigations or attempts to determine whether additional grout could be forced back of the shields after the completion of the first grouting?

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WIND-BRACING CONNECTION EFFICIENCY

Discussion

BY MESSRS. R. D. DALTON, AND JOHN J. GOULD

R. D. DALTON,[51] ESQ. (by letter).[51a]—The assumption that the force, R-S, in Fig. 3(c), is distributed as a triangular load on the length, C, is untenable. The surface upon which this force is presumed to be distributed is steel against steel and not steel against any other softer material. The overhanging length, C, is a cantilever and will take the form of a cantilever depending upon its load. It is inconceivable that the surface of the column steel would assume a shape such as to cause this distribution of load. It seems to the writer that the force, R-S, should be considered a concentrated load and that there must be some elongation of the rivet due to the lever action of the **T**-flange.

In Example 1, the assumption that the flange of the **T** acting as a beam is fixed at the rivet presumes an infinite force in the rivet or a deflection of the overhanging length, C, in the direction of the force, R-S. This deflection of the overhanging end seems impossible of attainment under the conditions.

In Fig. 7(b), if the heel of the upper angle is pulled away from the column face as indicated, the structural beam must have rotated about the point where the lower force, S, is applied, and the horizontal leg of the upper angle would have turned downward through the same angle with the top surface of the structural beam. Then, ϕ_1 does not equal ϕ_2 and Equations (5) and (6) do not hold true.

Also, if the structural beam is rotated, the horizontal leg of the lower angle is bent downward and the vertical leg of this angle is a cantilever. Therefore, the lower force, S, will be moved downward a certain distance, depending on the stiffness of the angle leg, and the value of Y will be somewhat greater than is given by Equation (7).

NOTE.—The paper by U. T. Berg, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: April, 1932, by Messrs. David Cushman Coyle, William R. Osgood, O. G. Julian, Harold S. Richmond, and Robins Fleming; and May, 1932, by Messrs. L. E. Grinter, C. R. Young, C. H. Sandberg, N. A. Richards, Jacob Feld, Dana Young, and A. J. Wilcox.

[51] Structural Engr., City Bldg. Dept., Oakland, Calif.

[51a] Received by the Secretary April 4, 1932.

In Equation (11), Δ_1 is the only Δ that needs to be considered since the structural beam is assumed to extend at least to the face of the column in moment equations, and the horizontal leg or stem of the connections is substituted for, and is approximately equal to, the immediately adjacent parts of the top and bottom flanges of the structural beam. If the horizontal parts of the connection are approximately equal in area to the adjacent parts of the top and bottom flanges, Δ_2 and Δ_4 are already taken care of in the I and L of the structural beam.

There is one action, where a continuous-slab floor is used, that may tend to relieve the bending in the connections due to gravity loads, and especially that due to live loads. When a beam is bent the tension side becomes longer and the compression side shorter. Therefore, if the slab will hold the columns a constant distance apart, a force is set up at the column face in line with the lower flange of the beam. (See Fig. 24.) Whether this action actually exists

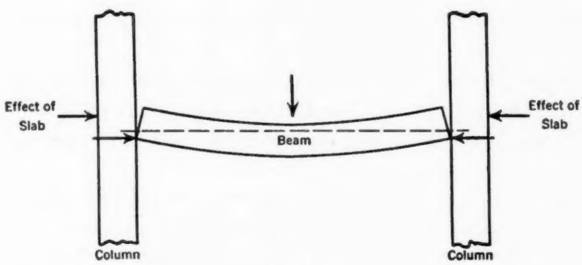

FIG. 24.

and, if it does, to what extent it may be depended upon, is very problematical and the idea is only presented here with the thought that it may or may not be found useful after investigation.

Lest this discussion appear too critical, the writer agrees with Mr. Berg in his conclusions that designers should not neglect the effect of gravity loads on the connections, and should strive for better balanced connections, and a better analysis of joints. Although he believes more logical basic assumptions might be made, the paper is a forward step in breaking away from the rule-of-thumb methods of the wrought-iron age.

Further tests on typical joints should be made with special reference to the action of connecting angles and **T**'s under stresses below the elastic limit. Tests beyond the yield point or to failure do not give much information on the distribution of moments in a structure if the members of the structure are working at a stress below the elastic limit.

JOHN J. GOULD,[52] ASSOC. M. AM. SOC. C. E. (by letter)[52a]—In the Synopsis of his paper the author states that where wind connections are provided, the effect of vertical loads on these connections as well as their adjoining members should not be neglected. It is common practice, however, to pay no atten-

[52] Structural Engr., with L. H. Nishkian, M. Am. Soc. C. E., San Francisco, Calif.
[52a] Received by the Secretary May 2, 1932.

tion to the effect of the vertical loads on the relatively rigid wind connections. Generally speaking, the structural engineer proceeds along the following routine.

Step 1.—Design of a Structure for Vertical Loads.—Beams and columns are designed as a statically determinate system; that is, the connections between beams and columns are assumed to be hinged and frictionless. The columns take vertical loads only; no moments are transmitted from the beams. The elastic behavior of the material according to Hooke's law is accepted.

Step 2.—Design of a Structure for Horizontal Loads.—Beams and columns are designed as a statically indeterminate system; that is, the connections between beams and columns are assumed to be rigid. The columns and beams take moments as well as shears. Hooke's law is accepted as under Step 1. It is usually assumed that the masonry takes no loads.

In comparing the fundamental principles underlying the design analysis as stated under Steps 1 and 2, the logic of the routine appears to be highly questionable.

Both analyses are based on Hooke's law that the strain is proportional to the stress. In one design for one kind of loading, a joint is assumed to be able to rotate freely, while in the next step for another kind of loading this same joint is converted into a rigid connection.

Some engineers are raising the question as to what is happening to this joint when gravity alone is at work, and are led to assume that the conflict between the different assumptions can be reconciled because steel has an additional quality, namely, ductility. It is also claimed that the steel frame adjusts itself.

In looking honestly into these problems the explanation is given that the "adjustment" referred to, takes place in the connections and that "ductility" implies that steel becomes malleable when subjected to high stress.[53]

The writer believes that at present the problems of adjustment of connections have been placed within reach of a sound analysis. Tests made by W. M. Wilson, M. Am. Soc. C. E., and Professor H. F. Moore,[54] have demonstrated that rigidly built connections fulfill their duty for any kind of loads if designed within the elastic limit. It is shown that errors on the frame due to adjustments are either negligible or can be properly analyzed and guarded against.

Ductility, however, still remains one of the magic forces. So far as the writer understands the present building practice, it is quite sufficient to overstress some particular part of a structure, generally the wind connections, make the steel malleable, thereby raising the elastic limit, and the structure will stand up.

The proponents of the ductility theory have not availed themselves, during the past half century, of the opportunity to clarify their case.[55] In order

[53] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1.

[54] "Tests to Determine the Rigidity of Riveted Joints," *Bulletin No. 104*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[55] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1213.

to take definite advantage of ductility in a design, systematic research, together with a rational theory, should form the basis of an analysis rather than speculative assumptions. Tests of connections, built according to the ductility theory, and subjected to reversible dynamic forces, are also suggested.

It appears to the writer that the major adjustment of a structure does not take place in the connections or in the property of the material, but in the actual factor of safety of a structure. In other words, if a typical structural steel frame were built without any load-resisting walls of fire-proofing; if it were then loaded vertically and horizontally to the extent of the assumptions and designed along the routine as outlined under Steps 1 and 2, it would have an actual factor of safety considerably smaller than that calculated.[56] Theoretically, such a frame would be inefficient and uneconomical because of its non-uniform distribution of stress and strength.

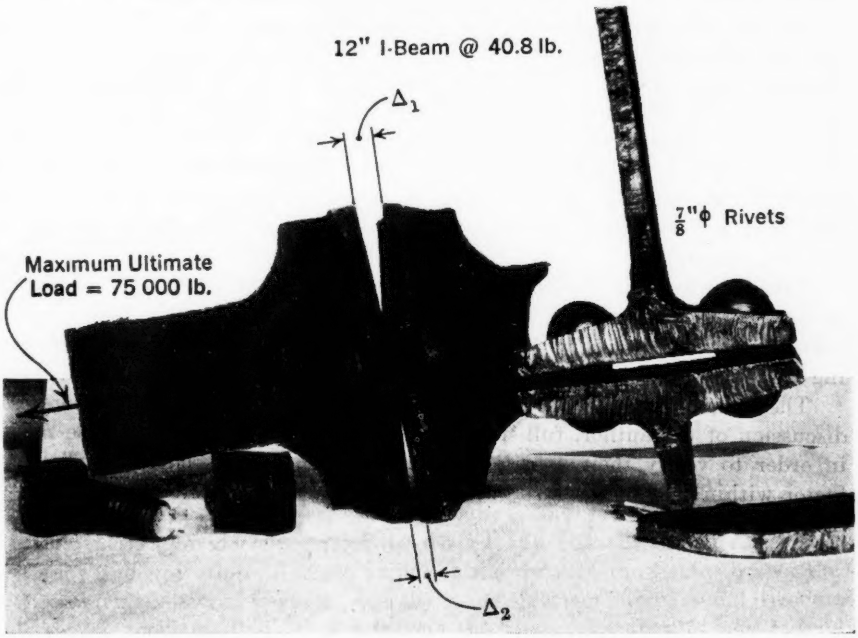

FIG. 25.

In regions where a structure is never called upon to deliver its ultimate strength capacity, this condition is apparently of little consequence to the investing public. It also does not seem to bother, seriously, the conscience of professional structural engineers.

The problem, however, becomes magnified in countries where either earthquakes or hurricanes are expected. In such countries the building public is vitally interested in maximum protection for a minimum cost. It should

[56] *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1485.

be apparent that the owner of a building is entitled to a structural frame in which the elastic limit of the material is not exceeded before the disaster occurs, and that a safety margin of a building should not be assumed in the steel frame while actually a great part of it is in the fire-proofing and masonry walls.

In the action of earthquakes on buildings one of the few definitely known facts is that a shock will produce bending moments and shears at the beam and column joints. In designing a steel structure as a continuous elastic frame for vertical loads, the savings made in the beams are placed in the connections and columns. This will make a structure far more immune from earthquake damage with about the same amount of investment, or very little more, than could be done according to the present design practice.

It is not intended to suggest that an engineer should apply to all structures a tedious analysis, but rather that he should be aware of the relative merits to the building owner of his fundamental assumptions.

As an addition to Mr. Berg's commendable paper, Fig. 25 illustrates a case in which the relative value of rivets and bolts in tension, of the same diameter, were tested. These tests were made by L. H. Nishkian, M. Am. Soc. C. E. Of all the tests made, the rivet had an ultimate strength of about 40% (without breaking) greater than the bolt. It is interesting, furthermore, to note that the half **I**-beam suffered great deformations in the direction parallel to the web (Δ_1 Δ_2, Fig. 25). Reference is also made to the author's Fig. 3(*b*), concerning which the statement is made that the flange cannot act as a cantilever.

Undoubtedly, this condition is true for most of the cases. It is possible, however, that where long rivets are used, the elongation of the rivet under normal stress condition, will prevent the outer edges of the flange from bearing on the column flanges. In these cases, the flange will act as a cantilever.

The writer would recommend that in conjunction with the theoretical discussion of the author, full-sized tests of half **I**-beam connections be made, in order to verify their safe loading capacity so as to keep their efficiency factor within that of the structure as a whole.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

PUBLIC SUPERVISION OF DAMS A SYMPOSIUM

Discussion

By Messrs. Harry W. Dennis, N. Kelen, Ford Kurtz, and William W. Tefft

Harry W. Dennis,[23] M. Am. Soc. C. E. (by letter).[23a]—The paper by Mr. Hinderlider presents a strong case in support of the proposition that dams are potential sources of danger, that they should be designed with skill and care, and that they should be reviewed by some independent agency. The supervision of the work by the State provides such review by an agency that is outside the domination of the owner and this is a strong argument.

Under "Legislative Trends," Mr. Hinderlider states that while sound principles have been developed in the design of almost all conservative types of dams, there is still some diversity of opinion regarding theories that affect the safety of huge masonry dams. This emphasis and that expressed in connection with the dam failures in Table 1, show that although the supervision by the State presupposes that the engineer "knows all about it," there are still many, many things in connection with the design and construction of dams that the engineer (speaking broadly) does not know as yet. If the State is to provide supervision of design and construction and of maintenance and operation of dams, is it not the duty of the State to provide the necessary additional research whereby its officials can carry on and can develop research problems that are far beyond the financial ability of any individual corporation, or technical society, or any agency except the public treasury?

Who knows what the behavior of large masses of concrete may be? It is well known that the temperatures created in the setting of concrete dissipate

Note.—The Symposium which includes the paper by A. H. Markwart, M. Am. Soc. C. E., presented at the meeting of the Power Division, New York, N. Y., January 16, 1930, and the paper by M. C. Hinderlider, M. Am. Soc. C. E., presented at the Technical Session, Sacramento, Calif., April 23, 1930, respectively, was published in January, 1932, *Proceedings*. Discussion of the Symposium has appeared in *Proceedings* as follows: March, 1932, by H. deB. Parsons, M. Am. Soc. C. E.; April, 1932, by Messrs. William P. Creager, M. M. O'Shaughnessy, N. A. Eckart, R. C. Johnson, F. W. Hanna, and Joel D. Justin; and May, 1932, by Messrs. I. C. Steele and Walter Dreyer, Fred A. Noetzli, George N. Carter, George W. Hawley, and H. K. Barrows.

[23] Chf. Civ. Engr., Southern California Edison Co., Los Angeles, Calif.

[23a] Received by the Secretary March 21, 1932.

over a long period of time, depending, of course, on the mass. In gravity dams in which the thicknesses are in excess of, say, 100 ft., the fact has been established that many years pass before the ineherent temperatures, due to the setting of the concrete, will have been entirely dissipated. Engineers do not know what is going on in huge masses of concrete.

If the State Engineer is to be the supervising official, with the necessary facilities to determine some of these things about which so little is known, his status and power in this connection should be established within the same legislation that creates the State as the supervising agency.

Effective legislation in connection with this subject seems to have required the invocation of the theory of the police power of the State. That, therefore, makes the legislation in the interest of the public safety. From that premise it follows that the only authority that can be given to the supervising officials, is that which comes in administering their duties in the interest of the public safety. Their authority is limited. The writer takes some exception, therefore, to Mr. Hinderlider's recommendation that the law should include authority to "pass on the design, construction, maintenance, and operation of all dams and reservoirs of a certain minimum height and capacity" (see Conclusion 1). Many of the recommendations are acceptable, if there is added after each one, "so far as the public safety is involved," but the writer cannot agree that the supervisory body should be given blanket authority to exercise control over these things regardless of whether or not they involve the public safety.

Consider the recommendation (Conclusion 6) that the law should include authority to exercise supervisory control during construction and operation of the dam following construction.

The financier who is asked to approve an application for the necessary funds for a new project examines the estimates of cost and cannot take exception to the condition whereby the State may impose changes in plans and specifications before the work is begun. Revised estimates may be made as a result of such changes and examined before the project is authorized. If, on the other hand, the State official has the authority to change those plans and specifications from time to time, at his own will, and for reasons other than the public safety, there will be difficulty in financing large and important structures.

Mr. Hinderlider's comments on Item V, of what State supervision should not include, contains the gist of the writer's attitude, namely,

> "State supervision should be limited to the approval or disapproval of the plans and data presented, and the authority to require essential amendments thereto, which, in the judgment of the State official, appear to be needful for insuring the proper degree of safety, * * *."

DR. ING. N. KELEN[24] (by letter).[24a]—Convincing proof of the necessity of the licensing and supervision of the design, construction, and operation of dams by the State is advanced in this paper. State supervision is necessary for two main reasons. In the first place hydraulic establishments are

[24] Cons. Engr.; Privatdozent, Technische Hochschule, Berlin-Charlottenburg, Germany.

[24a] Received by the Secretary April 8, 1932.

public utilities; each artificial interference with the natural stream flow touches public interests and can have consequences that are more or less noticeable both above and below it. For this reason in most countries the State is the proprietor of all bodies of water. In the second place a dam creates a certain menace to life and property in the region below it because, as correctly emphasized in the Symposium, dams differ from other engineering structures inasmuch as their failure may cause an enormous catastrophe. Some years ago it was estimated that the failure of a dam designed by the writer would involve the certain death of 10 000 people and (since densely populated industrial districts would have been destroyed by the ensuing flood wave) a property loss impossible to estimate. Considering such possibilities, the engineer hesitates to take over the sole responsibility for the structure especially since many points in theory, geology, and construction of dams are not yet quite clear. Mr. Hinderlider's Table 1, concerning the failures of 293 dams and their causes, is valuable and highly interesting.

If a central governmental bureau takes over the supervision of all dam construction beginning with the approval of the general design and including operation after completion, the technical safety of dams is guaranteed. Furthermore, it is possible for such a bureau to collect many data on the subject, which could be used successfully in further construction. The systematic collection and application of these experiences are necessary details that cannot be recommended too strongly. The writer could enumerate many instances in which the same errors were repeated because the engineer who designed the dam, or the manager of building operations, was not aware of the experiences already accumulated.

For example, it may be that in order to save money the geological examinations were not sufficiently thorough and, consequently, when excavation began the proper foundation could not be found or was found only at a greater depth than was expected, so that the costs for the dam were considerably higher than calculated; or, possibly, the dam site had to be reflooded and abandoned because the foundation was totally inadequate. Other cases have occurred in the writer's experience, in which the dam was finished and operation was begun, but in which it was impossible to raise the water to the necessary level because the reservoir was not tight enough and could not be made water-tight artificially. In these cases the result was merely a useless dam, an empty reservoir, and considerable sums of money were spent for nothing. Unfortunately, such experiences are gained only through one's own practice or indirectly, as in most cases the most valuable experiences of this nature are not published. Only a continuing State control would enable the collection, systematic use, and dissemination of information concerning the design, construction, and operation experiences of this nature. After all dams of a country have been placed under State control, it is possible to arrange an international exchange of experiences. Thus, every country is served and, finally, all mankind.

State control is also an absolute necessity with regard to a correct calculation of the flood-discharge capacity. The reliable calculation of the spillway requires years or even decades of systematic measurements of rain-

fall, water, sediment, and evaporation, and it is in the nature of things that these have to be made by State agencies. This is also necessary because all measurements must be based upon the same point of view. The correct dimensions of the spillway are among the most important requisites for the safety of dams. This is of special importance for earth dams, which are never allowed to be overflowed. In Table 1, Mr. Hinderlider shows that 159 of the 293 dam failures (that is, more than 54%), are earth dams, and the principal cause of such failures is found to be insufficient dimensions of the spillway; 44% of the earth dams failed for this reason.

State approval and supervision of dams are necessary in order that their construction may be made according to a consistent plan. Unregulated construction of dams can eventually impair the entire future development in rational use of streams and thus create considerable loss to the nation.

By requiring that measuring instruments be arranged in all newly constructed dams the State can contribute considerably to the safety of dams and the future development of dam construction. To clarify some of the moot questions that affect the safety and economy of dams, it will be necessary not only to install such instruments, but to arrange for continuous observation. The deformation of the structure must be observed constantly; the trend of the interior temperature must be followed; strain-gauges must be arranged for measuring stresses; hydrostatic uplift must be measured, etc. A sufficient number of such observations and measurements can clarify theoretical questions still pending, with the result that dams can then be erected more safely and more economically.

Of course, State supervision should not be allowed to lead to a bureaucratic rigidity and to hamper the free development of personal initiative. Under strict observance of the principles of safely, each person who has anything to do with design and construction must be privileged to make independent proposals, because this is the only way to promote a living technical agency. Out of their cumulative experience, the personnel of the central bureau for dam control will provide the facts that will furnish the basis of the real principles of safety.

One can conclude from the foregoing that it is reasonable to summarize these principles and to publish them as a building code. The opinions of colleagues differ as to whether or not such a code is necessary. Indeed, different opinions are justified by different local conditions. A small country, such as Switzerland, which can be easily surveyed and where conditions are rather homogeneous, can most probably dispense with a building code for dams. Larger countries, such as the United States, however, with greatly varying conditions, can scarcely do without such codes. They should be made uniform for all States, and such special factors as climatic conditions, earthquake hazard, etc., that typify the different parts of the States, can be easily taken into account in the specifications. It would be best perhaps to publish uniform regulations that are valid for all States and to issue supplements for States or State groups in which special conditions require them.

In Germany engineers have long been of the opinion that such regulations are absolutely necessary since they are of special importance, with

regard to national economy as well as to safety. The responsibility for their publication comes under the domain of the various States, because the dams are regulated by the various agricultural ministeries. Administratively, this may be justified, but technically it would be more reasonable to issue a code for the entire country. In effect, the new Prussian regulations for dams will be adopted as authoritative for all Germany. The old Prussian regulations are dated 1913 and consequently are antiquated, because dam construction, especially since the World War, has undergone a great, unprecedented development.

State control in Germany has been so effective that (although the dams do not have excessively large dimensions) thus far, not one structure has failed. There are fifty dams more than 25 m. (82 ft.) in height. Dimensions of a few of the more representative types are shown in Table 2. The highest are: The Bleiloch Dam (Item 1); the Edertal Dam (Item 2); and the Schwarzenbach Dam (Item 3). The highest earth dams are: The Sorpe Dam (Item 4); and the Söse Dam (Item 5). Thus far, Germany has no arch dams because the valleys are too flat and, therefore, unfitted for the erection of such dams. German engineers, however, recognize the fact that arch dams are the safest of all types. The Vöhrenbach Dam (Item 6, Table 2) is the only multiple-arch dam constructed to date.

TABLE 2.—DIMENSIONS OF CHARACTERISTIC DAMS IN GERMANY

Item	Dam	Total Height		Reservoir capacity, in acre-feet	Type	Date constructed
		In meters	In feet			
1	Bleiloch	65	213	174 000	Concrete	1931
2	Edertal	52	171	162 000	Masonry	1914
3	Schwarzenbach	67	219	12 000	Concrete	1925
4	Sorpe	69	226	66 000	Earth	1932
5	Söse	57	187	20 000	Earth	1931
6	Vöhrenbach	25	82	900	Multiple-arch	1924

In spite of this, the new regulations for dam building include all the principal types of dams, in anticipation of future development and they take into account all experiences that become available, from whatever source.

The German expression, "Talsperre," applies only to "storage dams." The others are called "Wehre," which is translated "overflow dams." The exact distinction between these two types causes great difficulties in practice and cannot always be made absolute. With regard to their construction movable dams are typical overflow dams. To avoid all possibility of misunderstanding, the Prussian Water Law defines storage dams as structures with a height above stream bed of more than 5 m. (16.4 ft.) and impounding more than 100 000 cu. m. (81 acre-ft.) of water. Other structures are considered as storage dams if their failure would involve considerable damage. This supplementary definition shows clearly the purpose of the regulations for dam building; that is, the guaranty of the public safety. Certain exceptions, especially regarding the foundation, are admitted for non-storage dams. Whereas a masonry dam must be founded on unyielding rock, masonry overflow dams may be founded on alluvial soil, sand, gravel, etc., with sufficient precautions.

It must be emphasized that the new Prussian regulations for dam building are not strictly specifications, but rather instructions. The official title is, therefore, "Instructions for the Design, Construction, and Operation of Dams." They were developed by a committee consisting of ten members, including the writer. The work required a session of fifty full days. While these instructions are all based on experience in Germany and elsewhere, the fact is stressed that they are meant neither as textbooks, nor as inviolable specifications, but only as instructions to which the officials who have the power of supervision and approval of the projects, may hold.

The instructions include thirty-five pages and are divided into the following eight main parts: Definition of a dam; preparation, form, and subject of the design; technical requirements; construction, operation, and maintenance by the owner; State supervision of construction and approval; State supervision of operation and maintenance; and example of instructions to the dam superintendent.

The technical requirements are the most voluminous, comprising twenty-six pages. The following types are considered: Gravity dams, arch dams, buttressed dams, and earth dams. These types are considered as main types; of course, the construction of other, newer, dam types is not excluded; it was only impossible to consider them because in the course of the last few years a number of newer types have appeared, the construction of which had to be judged separately in each case.

Regarding foundation conditions the "Code" stipulates that masonry dams must be founded on rock and it requires, in order to secure increased safety against sliding, that the toe of the dam be propped directly against a steep step of the solid rock. The usual grouting is recommended and instruments for the measurement of the hydrostatic uplift must be installed.

Arch dams are permitted only in mountains in which sites may be selected on geologically reliable, sound, and inflexible rock. For arch dams with a height of more than 30 m. (98.4) the "Code" recommends that the computations be checked by experiment with a model. Suggestions are offered for the calculation of stresses due to hydrostatic pressure, dead weight, temperature, shrinkage, etc. The highest allowable compression stress amounts to one-fourth the compression strength; and the maximum tension stress to one-third the tension strength. When reinforced concrete is used, the latter value is raised to one-half the tension strength. Compression and tension strength must be ascertained by 90-day test cubes. It is specified that vertical, radial joints be left open during construction and that they be grouted, after the principal shrinkage is complete, possibly in cold weather. In this way most of the disagreeable stresses caused by shrinkage and decrease of temperature can be avoided.

The "Code" also contains descriptions of calculations and detailed laboratory tests for the higher dams. Construction of earth dams by the hydraulic-fill method common in the United States is not mentioned in the instructions because German engineers are not quite convinced that dams constructed in this way are sufficiently safe.

FORD KURTZ,[25] M. AM. SOC. C. E. (by letter).[25a]—The writer wishes to record his complete agreement with the following salient ideas in Mr. Markwart's paper:

1.—There is necessity for introducing greater conservatism and sounder judgment, particularly the latter, in the design of dams;
2.—Critical defects in dams that have failed have been avoidable in many cases;
3.—Such avoidable defects can be eliminated almost entirely with effective regulation and control;
4.—With human nature constituted as it is, control from within is not to be expected;
5.—The value of State supervision will lie only in requiring an independent review of the design, thus insuring the application of more than one mind to the problem. The construction and operation of a great dam should never be left to the judgment of one man, no matter how eminent; and,
6.—Those in supervisory capacity on behalf of the State Government must obviously be the peers in technical experience and judgment of those designing and constructing the dam.

The requirements set forth in Statements (5) and (6), which are the indispensable elements of "effective regulation and control," can be met only through a permanent State Commission of competent engineers, not less than six in number, appointed by the Governor of the State and approved by the State Senate, the State Engineer to be an *ex officio* member of the Commission. Decisions should probably require a two-thirds vote, those dissenting being allowed to present minority opinions. All administrative and routine work would be carried out by existing agencies, but the absolute authority would rest with the Commission.

No useful purpose will be served unless the State Government brings to the solution of these problems men of skill equal to or greater than the skill of those employed by the owners. Therefore, it is of the utmost importance that authority be given to a commission rather than to a single individual, such as the State Engineer. The Commission must be chosen from competent men active and prominent in their profession. Obviously, such men cannot be precluded from practicing their specialty in the State which they serve, but the writer can see no reason why they need be thus precluded. If a judge is interested in a case directly or indirectly, he does not resign from the bench. He simply withdraws from his judicial post as far as that case is concerned. There is no reason at all why a member of the Commission should not act in the same manner when he is interested directly or indirectly in a project before the Commission.

Of the three features—design, construction, and maintenance—the first and last without question can be handled efficiently by a competent Commission. The second feature presents much greater difficulties and the "occasional inspection," which is suggested by Mr. Markwart, would not only be inadequate, but might be dangerous. Proper supervision of construction would require (at least on major projects) the residence of a Governmental

[25] Hydr. Engr., The J. G. White Eng. Corp., New York, N. Y.

[25a] Received by the Secretary May 12, 1932.

Inspector. Construction problems must often be settled quickly. Unless the Inspector is a man of qualifications equal to those of the Resident Engineer and the Construction Superintendent, his decisions will be of doubtful value.

Care must be taken to see that the Commission is not led insidiously into designing the dam. It must pass upon designs only and, if they are not approved, it must indicate the general basic reasons for disapproval. Details of methods of correcting disapproved features should be left to the owner's engineer.

"Effective regulation and control," such as outlined briefly herein, will be valuable to the owner, and therefore he should pay for it and pay liberally. Only by the payment of adequate compensation to members of the Commission and to the Resident Inspector can men of proper skill be induced to accept these duties. The public should pay some, but certainly not the major, portion of the cost.

As pointed out by Mr. Markwart, appeal to the Courts will always remain as the owner's right. That cannot be taken away from him. It would be his great safeguard against an incompetent or "political" Commission. It is not practicable to codify rules for the design, construction, and maintenance of dams.

WILLIAM W. TEFFT,[26] M. AM. SOC. C. E. (by letter).[26a]—In the ideal State every activity affecting the safety of life and property should, and would, be subject to State supervision. The ideal condition presupposes availability of infallible public servants having expert knowledge of the subjects under their supervision. If these ideal conditions could be attained, no doubt State supervision of dams would be beneficial.

Unfortunately, it will be difficult, if not impossible, even to approximate this under present political conditions. The great difficulty will be to find supervisors who have adequate experience and knowledge to fill the requirements and who, if found, would be willing to accept a subordinate position in some State Engineer's office. The safety of dams will not be guaranteed by entrusting supervision to "young Civil Service employees who may pass adequate examinations after coming freshly from universities, qualifying them to act as aides to the State Engineer."[26b]

Under the practical application of such an act, instituting public supervision of dams, plans prepared by some consulting engineering office having command of a lifetime of experience, would be placed under criticism, and subject to amendment, by junior employees having only occasional or book knowledge of the problems under consideration. This would not necessarily occur always, but undoubtedly it would happen in many cases. It would lead to disputes and delays and would tend to abandon altogether the construction of otherwise meritorious projects.

State engineers must exercise authority in many branches of engineering. It is possible, but not often the case, that the State Engineer is also an expert

[26] Cons. Engr., Vice-Pres., Fargo Eng. Co., Jackson, Mich. Mr. Tefft died June 24, 1932.
[26a] Received by the Secretary May 16, 1932.
[26b] *Proceedings*, Am. Soc. C. E., April, 1932, p. 686.

in the design and construction of dams. The problem would be referred to a special department in the State Engineer's office. Disputes would arise in which the promoter's or owner's engineers would be overruled by State authority.

Such conditions already exist in large corporations, and will be accentuated under State supervision. The Corporate or State authority, if desiring not to act on its own responsibility, is quite apt to call in some outside expert perhaps less familiar with the subject under consideration than its originators or designers, with the result that there are more delays, changes of unwarranted nature, extra expense, etc. In a recent case a $250 000 spillway was added by an outside expert and dictated solely by general considerations wholly inapplicable to the region in which the amended structure was located. It did not increase safety, and its cost was wasted.

Difficulties of supervision are increased by the present state of knowledge regarding the design of dams. A great many elements entering into such design are matters of experience, and wide diversity of opinion exists. Structures built for private undertakings are necessarily planned with the utmost economy. The expert consulting engineer who specializes all his life in dam design knows, or thinks he knows, how far economy in design can be carried without endangering the structure. The State supervisory body on the other hand would be inclined to be ultra-conservative, partly because of lack of long and exclusive experience, partly because the very object of its interference is to guarantee safety only. Economical considerations would not be the primary guide of the public supervisor. Disputes would be inevitable. Projects would be hampered and possibly entirely abandoned.

On the other hand, changes ordered for the purpose of increasing safety might have exactly the opposite effect, as previously mentioned. In a given State there might be two outstanding consulting engineers having experience with dams on earth foundation. One is widely known and in all probability would be chosen to supervise the dams of that State partly because of his political activities. He has designed eight or ten dams, two of which have failed. The other consultant has had to do with about seventy economically designed dams built mostly on earth, no one of which has failed. The point is that there is more risk to safety in choosing the supervisor than in the dams themselves as now being built throughout the United States. On a percentage basis the failure of dams is very small—less than for almost any other business in which money is invested or lives are jeopardized.

The design of dams is at present not an exact science. The National Electric Light Association appointed a sub-committee to gather "experience with different types of dams." This Committee compiled replies on a questionnaire pertaining to the problems involved in the design and construction of dams. The answers received from about twenty-six of the leading designers of dams in the United States showed the wide diversity of opinion among experienced engineers relative to many questions of a fundamental nature involved in the design of dams of various types.[27]

[27] Progress Rept. of Sub-Committee, Hydraulic Power Comm., National Elec. Light Assoc. (not yet published).

It will be quite impossible to formulate a State code for guidance in dam design, similar to the city building codes in existence. The principles followed in dam design are still very much in a state of flux; each individual case offers a variety of possible solutions. If a code cannot be made to cover the design of dams successfully, the critical examination of projects by a State Examiner will be based solely on personal judgment. This contains so many elements of uncertainty that the net effect will be to delay and hamper the successful launching of new projects.

Many projects for hydraulic power as well as irrigation are at present (1932) just within the "economic limit." Possible extravagant changes in design in behalf of unnecessary safety will cause them to be abandoned altogether. Dams for domestic water supply are more often able to bear additional cost that can be charged for in a higher rate for water. A large number of private irrigation and power projects, which have been in successful operation for many years, if required to be designed according to Government standards, would never have been built. The old Bear Valley Dam is a case in point.

The cry for governmental supervision of dam design is largely a matter of public hysteria, unwarranted on the basis of aggregate losses caused by failures among these structures. In the last fifty years there have been only two cases of dam failures that have attracted nation-wide attention, namely, the Johnstown and the St. Francis disasters. Such cases should not be permitted to recur, but State supervision can scarcely be expected to act as a guaranty. Under an ideal Government it might so act, but under existing political conditions in State Government, the public supervision of dams would merely impose additional hardships on a branch of enterprise already hard pressed by competitive and economic conditions, without offering the compensating guaranty of any better results than are now attained under private initiative.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

STEREO-TOPOGRAPHIC MAPPING

Discussion

By Messrs. R. E. Ballester, and T. P. Pendleton

R. E. Ballester,[23] Esq. (by letter).[23a]—This paper deals especially with map production by stereo-topographic methods and by photographs taken either from the air or from the ground. For the designing engineer who uses such maps the precision or accuracy that it is possible to attain with these methods is of primary importance.

The writer had the opportunity to order the mapping, by the stereo-topographic method from ground stations, of a stretch of river with steep slopes, and to compare the precision attained with that of precise leveling with the usual topographic instruments.

The area mapped covered 28 hectares (69 acres) at the Primero River at Córdoba, Argentine Republic.[24] The map was to be used for design purposes or for correction of the location of a dam and appurtenant works. The extreme difference in elevation was 100 m. (328 ft.). The work was performed by the "Instituto Geográfico Militar" (Army Department of Argentina). While the field work was prepared and executed to make a map on a scale of 1:1 000, the final map was traced with the stereo-comparator to scale 1:500, with contour intervals of 2.00 m. (6.56 ft.).

An independent field party staked out three profiles crossing the river at the proposed site. These were compared with profiles taken graphically at the same location using the stereo-topographic map. Comparing the three sets of profiles, from direct leveling and from the map, the writer derived the following values, in metric units, for the mean error in altitude and in plan:

$$m_h = \pm\,(0.06 + 0.49 \tan \alpha)$$

Note.—The paper by C. H. Birdseye, M. Am. Soc. C. E., was presented at the meeting of the Surveying and Mapping Division, Sacramento, Calif., April 24, 1930, and published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1932, by Messrs. Theron M. Ripley, O. S. Reading, and Lowell O. Stewart; and May, 1932, by Messrs. F. H. Peters, W. H. Crosson, Dwight F. Johns, and Douglas H. Nelles.

[23] Prof. of Applied Hydraulics, Univ. of Buenos Aires, Buenos Aires, Argentine Republic.

[23a] Received by the Secretary April 9, 1932.

[24] A detailed description is given in *La Ingenieria*, Buenos Aires, Vol. 35, No. 7, p. 222, July, 1931.

and,

$$m_p = \pm (0.49 + 0.06 \tan \alpha)$$

in which, m_h is the mean error in altitude; m_p, the mean error in plan; and, α, the slope of the ground.

Taking for α, the mean value of 35°, corresponding to a slope of 1 on 1.5, the mean error is: $m = \pm 0.41$ m. and $m \pm 0.57$ m. both for the 1:500 scale.

This precision was quite sufficient and satisfactory for the design of the dam, spillways, plant layout, etc. The writer believes that a topographic survey with transit and level would not have resulted in the same detail and precision, especially in view of the fact that slopes of the river gorge were so steep as to be almost inaccessible to the rodmen.

It will be interesting to learn the author's opinions concerning the precision attainable by the different methods of stereo-topographic mapping, for different scales. This will encourage engineers who cannot take the time to study stereo-topographic mapping in detail to select this method, which, in many instances, presents advantages of cost, time, and accuracy over the usual topographic mapping.

T. P. Pendleton,[25] M. Am. Soc. C. E. (by letter).[25a]—A new method of mapping is described in some detail in this paper. It calls for consideration on the part of engineers and others having need for topographic maps, and it may be that increasing familiarity with it will aid in clarifying an unfortunate situation that exists.

Consideration of Colonel Birdseye's paper will reveal that the introduction of stereoscopic methods of mapping into modern practice emphasizes the fact that topographic mapping is a task for the specialist. The principles on which such methods depend have been well developed, and although the various instruments differ widely in their manner of solving the problems involved, it can be stated that when properly used they are capable of producing excellent maps. The cost of the equipment is relatively high as none but the highest grade of workmanship can be tolerated and this quite effectively bars the small engineering organization from possession of instruments of this type. The method also demands personnel trained in its use and, at present, such men are not generally available.

A few statements regarding the differences in the American and European point of attack on the problem may not be out of place. The aerocartograph, which is described in considerable detail, has many points of resemblance to some other European stereoscopic mapping instruments. It combines in itself all the functions necessary to transform information contained in the aerial photographs to map form. This makes for an instrument of considerable complexity, with many optical and mechanical parts in motion when it is in use. Thus far, the only American method in use was devised with the idea of breaking up the work into its various essential steps and designing for each

[25] Washington, D. C.

[25a] Received by the Secretary April 14, 1932.

one a simple rugged instrument with few moving parts. It was felt that this method of attack was more satisfactory from a production point of view as it would enable the production engineer to localize causes of delay readily and speed up the output of that operation by assigning additional men for the purpose. It also assured freedom from instrumental troubles caused by wear and the need for frequent adjustment.

Another essential difference is in the character of the various optical parts. It is desirable that the two objective lenses in the aerocartograph match closely in their characteristics the lens used in the aerial mapping camera. When this is accomplished any faults in the camera lens are largely compensated for by similar faults in the objective lenses of the aerocartograph. This frees the manufacturer of the necessity of securing lenses highly corrected in the matter of distortion and enables him to use objectives of a quality readily obtainable on the market. On the other hand, the process introduced by Messrs. Arthur and Norman Brock makes use of lenses in the aerial cameras and office instruments that are selected largely for their freedom from distortion. Consequently, every photograph made is practically a true perspective projection of the original and no compensation for errors in the original photographs is necessary.

The matter of lens perfection must be considered in connection with the type of photographs to be used. It is quite useless to seek lenses in which the distortion has been eliminated if the aerial negatives to be used are not true point-to-point representations of the earth's surface. The only method thus far devised of grasping and retaining such a view is in the use of glass-plate negatives on which any motion of the emulsion that may occur is so slight that it can be ignored. It is for this reason that the Brock process makes use of glass plate negatives rather than roll film. Thus far, it has not been possible to coat a film base that would retain its shape satisfactorily. Attempts have been made with more or less success to secure a base that would change uniformly with changing atmospheric conditions and it is probable that some improvement has been made in this regard. No information has come to the writer, however, to indicate that success in this direction has been sufficient for the purpose. It is probably true that for mapping work in which the accuracy requirements are not great, aerial film negatives can be used satisfactorily, but at the most it is scarcely to be hoped that they will excel glass negatives in so far as freedom from change in shape is concerned.

Since aerial photography will play an important part in mapping methods of the future, it introduces another controversial point into the discussion of what constitutes a good topographic map. There is a wide divergence of opinion on the part of engineers and the faculties of technical colleges on this subject. Indeed this difference of opinion extends further, as there is the same lack of agreement as to the manner in which the actual mapping surveys should be carried out and the type of instrument best adapted to the purpose. Many colleges and universities give instruction in plane surveying and geodesy, but only a few devote enough time to these subjects to enable the student to judge of the merits of the various mapping methods.

Many and varied questions are continually arising which demand for their proper solution information that only a topographic map can give. These problems may be entitled to no greater weight, however, than many others involved in some major engineering study. The engineer in charge is frequently at a loss to know what type of map is most suitable and the method to be followed in making the necessary surveys. This is not surprising, as topographic mapping, in common with many other branches of civil and military engineering, is a specialized subject demanding instruction not commonly given in colleges, and which the engineer engaged in general practice has but little chance to acquire. It frequently happens that when the need for the map is felt a transitman will be diverted from his usual work of staking out structures, or of subdividing real property, and will be assigned to map work calling for training only acquired by a topographic engineer. As a result, he uses what instruments are at hand regardless of their suitability for the purpose and applies methods differing but little from what he would use in performing his usual work.

The result is frequently unsatisfactory because the map suffers from improper choice of methods and a lack of understanding on the part of the engineer as to what constitutes a good map. Sometimes, it suffers from faults that can be traced to false economies and, frequently, it must bear the burden of unnecessarily high cost due to the employment of untrained personnel and improper equipment and procedure. Skill in the use of the proper instruments and the development of what may be termed the engineer's topographic sense can only be acquired by working with and subject to the friendly criticism of men who have developed this ability to a high degree. If personnel of this training is not available it is advisable to request the aid of engineering companies making a specialty of topographic mapping, that are properly equipped and in a position to decide on the best method for any particular case, whether it be by one of the new stereoscopic methods or by methods with which all are familiar.

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DISCUSSIONS

DETERMINATION OF PRINCIPAL STRESSES IN BUTTRESSES AND GRAVITY DAMS

Discussion

By I. M. Nelidov, Assoc. M. Am. Soc. C. E.

I. M. Nelidov,[21] Assoc. M. Am. Soc. C. E. (by letter).[21a]—A development of the method of computing stresses in hydraulic structures is presented in this paper. It consists essentially of evaluating certain coefficients of a polynomial stress function and of solving for the value of this function with various values of x. The author deserves special credit for the application of this method to dams curved in plan and to tapering buttresses. His method is based on the linear distribution of one of the three unknown stresses.

The problem of the internal stress distribution arises mostly in connection with the following structural members, of which the cross-sections are large in relation to the length: (1) Beams fixed at one end, or cantilevers; (2) beams fixed at both ends; and (3) beams restrained at both ends, either straight or curved. The first class will be treated in the present discussion.

The writer wishes to complete the author's study by outlining briefly the present condition in the art of stress determination in gravity dams and also by arriving at equations developed by the author from the general equations of equilibrium.

Equations for stress distribution in cantilevers, triangular in shape and loaded with a uniformly increasing pressure beginning at the crest, were derived by M. Levy[22] and by P. Fillunger.[23] For the uniformly increasing load indicated, the stresses are linear as shown also in the author's Examples 1 and 3. The stress distribution in the cantilever of triangular and trapezoidal shapes for the uniform and uniformly varying pressures was determined by

Note.—This paper by W. H. Holmes, Assoc. M. Am. Soc. C. E., was published in January, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1932 by Messrs. W. J. Stich, Hakan D. Birke, Dirk A. Dedel, Fred A. Noetzli, and Eugene Kalman.

[21] Senior Engr. of Hydr. Structure Design, State Dept. of Public Works, Sacramento, Calif.

[21a] Received by the Secretary April 30, 1932.

[22] Comptes Rendus de l'Academie des Sciences, Vol. 127, p. 10, 1898.

[23] *Zeitschrift für Mathematik und Physik*, 59–60 Band, 1910–12.

Professor B. Galerkin.[24] The formulas for the trapezoidal profile are very complicated, so that for practical purposes the linear stress distribution may be used. When the sides of the cantilever are not parallel, or when its profile is curvilinear, the linear stress distribution for one of the three unknown stresses is the only one that can be used. In all the foregoing cases the influence of the foundation on the stress distribution is disregarded.

The general equations of equilibrium of an element within a homogeneous body of a thickness, t, in rectangular co-ordinates are:

$$\frac{\partial s_y}{\partial y} + \frac{\partial v}{\partial x} - w_c\, t = 0 \qquad (118)$$

and,

$$\frac{\partial s_x}{\partial x} + \frac{\partial v}{\partial y} = 0 \qquad (119)$$

in which, w_c is the unit weight of the element acting parallel with the vertical axis, Y; $t = f\,(x, y)$; s_y is the vertical normal stress; s_x, the horizontal normal stress; and, v is the shearing stress—all for the total width of the body.

Normally, a third equation of compatibility should be added in order to solve for the three unknown stresses. With the assumption of linear stress distribution for one of the unknown stresses the compatibility equation is not satisfied and the resulting solution for stresses becomes only approximate.

Integrating Equations (118) and (119) and neglecting the stresses along the plane, $X\,O\,Y$:

$$v = q t = -\frac{\partial}{\partial y}\int s_y\, dx + \int w_c\, t\, dx + f_1(y) \qquad (120)$$

and,

$$s_x = p' t = \frac{\partial^2}{\partial y^2}\iint s_y\, dx - \frac{\partial}{\partial y}\left[\iint w_c\, t\, dx + \int f_1(y)\, dx\right] + f_2(y) \qquad (121)$$

The values of the functions, $f_1(y)$ and $f_2(y)$, are obtained from boundary conditions. If the stress, $s_y = pt$, is a comparatively simple function of x and y, Equations (118) and (119) can be integrated directly; if not, the approximate integration must be used. For the linear stress distribution for s_y, Equations (120) and (121) become:

$$v = -\frac{\partial}{\partial y}\int\left[\frac{P}{a} - \frac{M}{I}\,(i - x)\right] t\, dx + \int w_c t\, dx + f_1\,(y) \qquad (122)$$

and,

$$s_x = \frac{\partial^2}{\partial y^2}\iint\left[\frac{P}{a} - \frac{M}{I}\,(i - x)\right] t\, dx - \frac{\partial}{\partial y}\left[\iint w_c t\, dx + \int f_1\,(y)\, dx\right] + f_2\,(y) \qquad (123)$$

[24] "On the Problem of Stresses in Dams and Retaining Walls with Trapezoidal Profile," by Prof. B. Galerkin, reprint from *Proceedings*, Leningrad Inst. of Ways and Communications, 1929.

In these equations, i is the distance from the origin of co-ordinates to the center of gravity of the base section.

For the approximate integration, p stresses should first be computed on three consecutive planes. Then Equations (122) and (123) becomes:

$$v = \frac{1}{2\,\Delta y}\Bigg[\left(\frac{P}{a}\int t\,dx\right)_u - \left(\frac{P}{a}\int t\,dx\right)_d + \left(\frac{M}{I}\int (i-x)\,t\,dx\right)_d$$

$$- \left(\frac{M}{I}\int (i-x)\,t\,dx\right)_u + \frac{1}{2}\left[\left(\int w_c t\,dx\right)_1 + \left(\int w_c t\,dx\right)_2\right]$$

$$+ \frac{V_1 + V_2}{2} \qquad \ldots\ldots\ldots\ldots (124)$$

and,

$$s_x = \frac{1}{\Delta y^2}\Bigg[\left(\frac{P}{a}\iint t\,dx\right)_u - 2\left(\frac{P}{a}\iint t\,dx\right)_o + \left(\frac{P}{a}\iint t\,dx\right)_d$$

$$- \left(\frac{M}{I}\iint t\,(i-x)\,dx\right)_u + 2\left(\frac{M}{I}\iint (i-x)\,t\,dx\right)_o$$

$$- \left(\frac{M}{I}\iint (i-x)\,t\,dx\right)_d\Bigg]$$

$$+ \frac{1}{\Delta y}\left[\left(\iint w_c t\,dx\right)_1 - \left(\iint w_c t\,dx\right)_2 + \frac{V_1 + V_2}{2}\right] + F \ldots (125)$$

After substitution of all the numerical data and consequent integration Equations (124) and (125) become polynoms with numerical coefficients and with x as an independent variable. These polynoms are exactly the same as those derived by the author.

The first of the following two examples demonstrates the case in which the direct integration is possible. The second example is made with the aid of the approximate integration.

Example 1.—Using the data in the author's Example 3, let the ratio of buttress spacing to buttress thickness be $J_1 = 12$. Then (in pounds per square inch):

$$\frac{P}{a} = \left(1 + \frac{J_1\,n}{k\,(n+m)}\right)\frac{w_c\,y}{2} \times \frac{1}{144} = J_2\,y = 3.125\,y$$

$$\frac{M}{I} = \frac{\frac{1}{k}\,J_1\,(2 - 3\,m\,n - n^2) + (n^2 - m^2)}{(n+m)^3} \times \frac{W_c}{144} = J_3 = 6.252$$

$$i = \frac{m-n}{2}\,y = J_4\,y = -\,0.5\,y$$

$$q = -\frac{\partial}{\partial y}\left[(J_2 - J_3\,J_4)\,y\,x + J_3\,\frac{x^2}{2}\right] + w_c\,\frac{x}{144} + f_1\,(y) = -\,5.25\,x$$

and,

$$p' = -\frac{\partial}{\partial y}\int q\,dx + f_2\,(y) = 0$$

Example 2.—In a slab and buttress dam (Fig. 26) let the vertical face be 1 ft. thick; the down-stream batter, 0.7; the water surface at the crest; the height, 20 ft.; the spacing of the buttresses, 12 ft.; the thickness of the

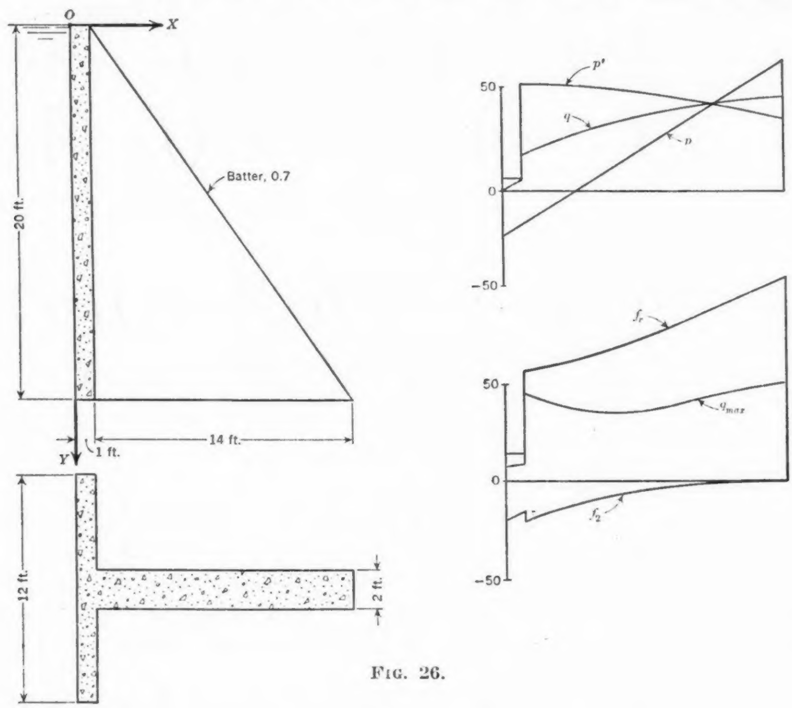

Fig. 26.

buttresses, 2 ft.; the unit weight of concrete, $w_c = 144$; and the unit weight of water, $w_w = 62.5$.

By ordinary calculations the stresses (in pounds per square inch) are: At the up-stream face, $p = -22.1$, $q = 0$, and $p' = 8.7$; and at the down-stream face, $p = 69.5$, $q = 48.6$, and $p' = 34.1$. Using Equations (124) and (125), the following expressions for stresses are obtained:

For $x < 1$ ft.: $q = 3.28\,x - 0.0738\,x^2$, and $p' = 8.71 - 0.0831\,x^2 - 0.0017\,x^3$.
For $x > 1$ ft.: $q = 16.05 + 3.328\,x - 0.0738\,x^2$, and $p' = 51.92 - 0.0831\,x^2 + 0.0017\,x^3$.

Table 3 gives the values of the stresses and the computed principal stresses.

TABLE 3.—Principal Stresses, Example 2.

x	p	q	p'	f_1	f_2	$q_{max.}$
0..........	—22.1	0.0	8.7	8.7	—22.1	15.4
1..........	—16.0	3.2	8.6	9.0	—16.4	12.7
1..........	—16.0	19.3	51.8	56.9	—21.1	44.9
5..........	8.5	30.6	50.1	66.3	— 7.7	37.0
10..........	39.1	40.9	45.3	83.3	1.2	41.0
15..........	69.5	48.7	35.0	103.9	0.0	52.0

It follows from the foregoing computations that the point at which the slab and the buttress join is dangerous from the stress viewpoint. The stresses at the "neck" change to high values rapidly. This is especially true of the case with the inclined face. Therefore, either reinforced haunches should be provided or the sliding support for the face must be used. This also shows the importance of the investigation for the internal stresses with the structures having sudden changes in the area of the base.

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DISCUSSIONS

WIND-BRACING IN STEEL BUILDINGS

SECOND PROGRESS REPORT OF SUB-COMMITTEE NO. 31, COMMITTEE ON STEEL, OF THE STRUCTURAL DIVISION

Discussion

BY MESSRS. DAVID CUSHMAN COYLE, S. P. WING, A. H. FINLAY, J. D. GEDO, V. A. VANONI AND M. P. WHITE, AND H. V. SPURR.

DAVID CUSHMAN COYLE,[28] M. AM. SOC. C. E. (by letter)[28a].—The following comments on the stiffness of tall buildings may help to illuminate some of the questions put forward on that subject by the Sub-Committee.

The relation between frequency of vibration and the weights and known characters of all the New York buildings tested, indicates that the fundamental note is by far the the most intense of the various frequencies. For practical purposes the building acts like a simple cantilever combined with a simple drifting frame. The elastic curve apparently has no nodes in it, but its real shape is beyond the power of mathematics to unravel. Tests show, as one would expect, that there is a sharp break in the curve at a large step-back and that the relative influence of the cantilever and drifting action is a matter of joint stiffness, accurate distribution of drift, effect of masonry, and similar factors which can only be treated mathematically with "the tongue in the cheek".

The general formula of harmonic motion can be put in the form, $t^2 = \frac{\pi^2 r m}{f}$, in which, t is the time of a single one-way swing; r is the displacement at extreme distance from the center; and m is mass to which the force,

NOTE.—The report of Sub-Committee No. 31, Committee on Steel, of the Structural Division, on Wild-Bracing in Steel Buildings, was presented at the meeting of the Structural Division, New York, N. Y., January 21, 1932, and published in February, 1932, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: May, 1932, by Messrs. L. J. Mensch, Robins Fleming, Rudolph P. Miller, C. M. Goodrich, Albert Smith, Hugh L. Dryden, L. E. Grinter, P. L. Pratley, Frederick Martin Weiss, and A. J. Hammond.

[28] Cons. Engr., New York, N. Y.

[28a] Received by the Secretary April 12, 1932.

f, is applied to produce a static deflection, r. The following relations among the quantities are useful in considering general features of construction ($\frac{f}{r}$ is the stiffness; $\frac{1}{t}$ is the frequency; and m is proportional to the weight):

(1) The frequency is proportional to the square root of the stiffness, and varies inversely as the square root of the weight.

(2) For a given weight, the frequency varies as the square root of the stiffness, and amplitude varies inversely as stiffness; therefore, the maximum acceleration in a given wind, being proportional to amplitude times frequency squared, is independent of stiffness.

(3) Amplitude is independent of weight; therefore, the only way to reduce the acceleration in a given design is to add weight near the top. This will reduce the frequency without increasing amplitude.

(4) Amplitude is inversely proportional to stiffness, and directly proportional to applied force.

(5) Frequency is independent of applied force.

Comparison of Buildings of Different Sizes.—Consider the relative deflections of two buildings of the same shape, one being k times as high as the other, both subject to a wind force per square foot of 0 at the ground, and

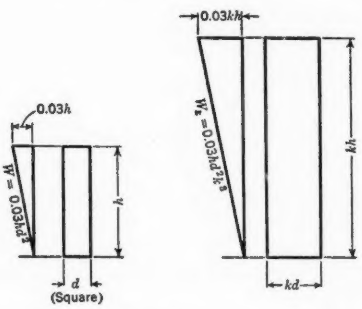

FIG. 11.

FIG. 12.

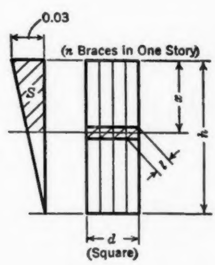

FIG. 13.

0.03 h at any height, h. Three criteria of comparison may be established, as follows:

(A) Regard the tower as a uniform mass of partitions and walls, with a base, d^2 (see Fig. 11); then, $I = \frac{d^4}{3}$; $d_k = kd$; $I_k = k^4 I$; $h_k = kh$; the maximum deflection, $D = \frac{11\,W h^3}{60\,E I}$; $W_k = k^3 W$; and,

$$D_k = \frac{11\,k^3 W\,k^3 h^3}{60\,E\,k^4 I} = k^2 D \qquad (3)$$

(B) Regard the tower as a cantilever with windward and leeward columns acting as flanges (see Fig 12). Let n = number of columns; A = area of

each column at the base; and $n_k = kn$, nearly. Then, $A_k = kA$; $I_0 = \frac{nAd^2}{2}$; $I_{0k} = \frac{knkA\,k^2d^2}{2} = k^4 I_0$; $D = \frac{Wh^3}{4\,I_0 E}$; $W_k = k^3 W$; $h_k = kh$; and,

$$D_k = \frac{k^3W\,k^3h^3}{4\,E\,k^4 I} = k^2 D \quad\text{.....................(4)}$$

(C) Regard the tower as in shear acting on diagonal braces all at the same unit stress per square inch (see Fig. 13); then $n_k = k^2 n$; S = shear $= 0.03\ hd\ (x - \frac{x^2}{2h})$; f_s = stress in braces; D_l = drift in one story $= \frac{s}{n}\frac{f_s l}{E}$ secant angle; D_x — drift per foot height $= \frac{s}{n}$ (constant); $D = C_1 \frac{0.03\,hd}{n} \int_0^h (x - \frac{x^2}{2h})\,dx = C_2 \frac{0.03\,h_k d_k h^2}{n}$; and,

$$D_k = C_3 \frac{kh\,kd\,k^2h^2}{k^2 n} = k^2 D \quad\text{.....................(5)}$$

Therefore, it may be assumed roughly that if two towers are the same shape and type of construction, their deflections will be about in proportion to the squares of their heights, unless special measures are taken to prevent it.

The force applied to 1 sq. ft. of surface facing the wind at the top is f, which is proportional to the height. The mass acted upon is proportional to d, which is proportional to the height, in this case. Hence, stiffness $= \frac{f}{D}$; $f_k = kf$; $D_k = k^2 D$; $\left(\frac{f}{D}\right)_k = \frac{1}{k}\left(\frac{f}{D}\right)$; $\left(\frac{r}{f}\right)_k = k\left(\frac{r}{f}\right)$, since r is proportional to D; mass $= m$; $m_k = km$; $t^2 = \frac{\pi^2 r m}{f}$; and,

$$t^2_k = \frac{\pi^2 kr\,km}{f} = k^2 t^2 \quad\text{.....................(6)}$$

Therefore, the frequency, $\frac{1}{t}$, is inversely proportional to the height, and the amplitude varies as the square of the height; acceleration (frequency, squared, times amplitude) is independent of the height; and frequency times amplitude varies as the height. This indicates that if sensation depends on acceleration, it will be no greater in a tall building than in a short one, provided the plan dimensions are in the same proportions as the heights; but if sensation varies as frequency times amplitude, which seems more probable, then heavier dead loads will have to be used as a means of reducing frequency, if towers are ever to go to greater heights than at present.

If it is assumed that the plan of the tower remains constant and the height is increased in the ratio, k, then regarding the tower in the three aspects (A), (B), and (C), the deflection of the high tower will be k^5, k^4, k^3, respectively, times that of the lower. As the true behavior of the tower is an unknown combination of all three, the exact value of the exponent cannot be determined; but it indicates that as towers increase in height without a corresponding increase of base line, the deflection increases as, say, the fourth power of the height. Frequency, by the same reasoning, would be inversely

proportional to the square of the height, so that acceleration, as before, is not increased. Frequency times amplitude, however, in this case varies as the square of the height, which may become the significant relation between allowable absolute height and slenderness ratio.

The crucial unknown facts at present are the form of the function of frequency and amplitude which corresponds to intensity of sensation, and the statistics of existing structures, good and bad, which will serve to mark out the threshold of sensation for different kinds of occupancy. Both these questions have been the subject of considerable discussion, and the necessary research will be undertaken as funds become available.

S. P. Wing,[29] M. Am. Soc. C. E. (by letter).[29a]—The writer regrets that the members of the Sub-Committee were unconvinced by the discussions of the First Report as to the necessity of changing their recommendations to include different wind loading for different localities, different pressure coefficients for different shapes of buildings, and increased loadings for the New York area. As the Sub-Committee has asked for additional information, herewith are submitted some rough studies of material taken from the U. S. Weather Bureau records. The data used and their treatment are by no means exhaustive, but it is believed the results are of sufficient value to warrant a change in the Sub-Committee's recommendations.

Requirement of Code.—A code controlling the design of wind-bracing has as its first essential the specifying of such loadings that if adequately met in the design, by no matter what means, absolute structural security to life of tenants and property of others is assured. The matter of uncomfortable vibration or slight permanent distortion of a building, while of first importance to the owner, affects the public only in a secondary manner. Structural security should not be allowed to depend on a secondary specification controlling rigidity.

Modern practice seems agreed that design loads, including impact where necessary, shall be the maximum that can conceivably come on a structure and shall be considered apart from the unit stress; and that, having assumed these loads, the unit stress may be chosen as high (that is, as near the elastic limit) as the character of the material and complexity of the structure will permit, without any allowance for possible increase in load. In determining loads, therefore, whenever factors are in doubt, the decision must lean toward the high side.

Discussion of Wind Data.—For wind loads, all basic available information comes in the form of numerous velocity records kept by the Weather Bureau. (Throughout this discussion true and not recorded wind velocity is used, unless otherwise noted.) The determination of maximum load involves the determination of the maximum wind velocity likely to be experienced by a building during its life.

The Weather Bureau anemometers are of the recording Robinson cup type and are usually mounted on the tops of buildings on a light, angle

[29] Civ. Engr., U. S. Bureau of Reclamation, Denver, Colo.

[29a] Received by the Secretary April 19, 1932.

framework, 15 to 20 ft. above the top of the roof. The cups of the instrument are mounted on a vertical axis and revolve in a horizontal plane.

Unfortunately, the wind around a building does not in general flow in horizontal stream lines. It strikes the building, curves over the roof, and may strike the anemometer at an angle. If so, the anemometer will under-register the true velocity by a function of the vertical angle. On the other hand, the wind in flowing in a curve usually increases in absolute velocity, as compared with the unobstructed wind stream. Tests on a model in a wind tunnel[30] showed an increase of 20% in one case. Such an increase may be counterbalanced by the characteristic of the instrument previously mentioned, but there is no definite relationship between the two conditions.

An important factor affecting recorded results is the inequality of exposure of the instruments. An obstruction located five or six times its diameter from the anemometer will affect the reading of the instrument. Few stations have an exposure that is satisfactory in this respect. In examining Chicago, Ill., records for the present study, it was found that in a shift of stations, although the anemometer was raised from Elevation 274 to Elevation 310, the average high wind velocity was reduced from 53.5 to 43.5 miles per hour. Investigation showed that this probably was due entirely to a shielding in the direction of the high winds.

During the life of the anemometer at one location increasing heights of the adjacent buildings will cause a change in the exposure and hence one year's record is not comparable with another. In addition (fortunately for this study), there has been in most cities a shifting of the anemometer station from one building to a higher one as the city has grown. To make records comparable it becomes necessary to correct for this change. It is to be regretted that, when these changes of elevations were made, duplicate records were not kept at both stations for a short time, so that a correlation of the two records might be made. It is suggested that the Sub-Committee make an endeavor to have this done in the future.

The consequences of all the foregoing facts are that most of the velocity records in existence are subject to considerable uncertainty and probably do not give a true picture of the unobstructed wind stream at a given station. The uncertainty may be of the order of 15% and there is nothing to show whether it is plus or minus.

Frequency Studies.—To the writer the question of wind velocities at a given location seems much like a study of maximum floods. Certain areas are found to be in the paths of high winds and are more subject to them than others; and, at a specific locality, the magnitude of the maximum wind for a given term of years appears to be a matter of chance. The records of the Weather Bureau for the stations at New York, Chicago, Pittsburgh, Pa., and Point Reyes, Calif., have been roughly analyzed. Fig. 14 shows the distribution of the maximum yearly 5-min. winds for those stations at certain elevations, plotted on frequency paper. These curves suggest characteristics of winds which further study confirms.

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[30] *Engineering News-Record,* January, 1932, p. 138.

It is apparent that there is a great difference in the expectancy of a given wind at a given station. Almost every year the station at New York experienced a higher wind than had ever been registered at Pittsburgh. Both stations were approximately the same elevation above ground. At Point

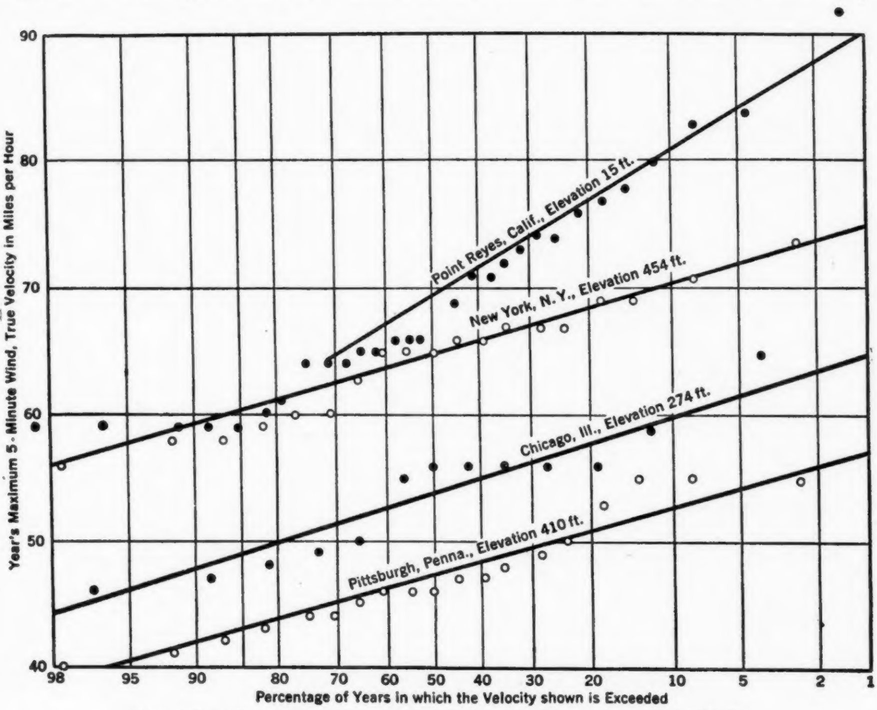


FIG. 14.—DISTRIBUTION OF MAXIMUM YEARLY 5-MINUTE WINDS.

Reyes, although the anemometer was near the ground, the velocities were greatly in excess of those at New York, 440 ft. higher. It also appears that the velocities at Point Reyes close to the ground were more variable than those at higher elevations elsewhere.

On Fig. 15 a more detailed study of these stations was made, using the data for each different elevation of the anemometer at each station. The mean of the maximum yearly 5-min. winds at each different elevation is used as a base and each year's wind is plotted as a percentage of this, making all the records directly comparable as to variability.

Examination of these curves shows that the 1% wind, or the wind that is likely to occur on the average once per 100 years, is about 130% to 135% of the term mean for the stations at the lower elevations, whereas it is only 120% to 125% at the higher elevations. This illustrates a fact previously suspected, that the winds at higher elevations are less variable than those near the ground.

The regularity of the Point Reyes record of 30 years is noticeable and lends support to the view that freedom from local interference and a long-

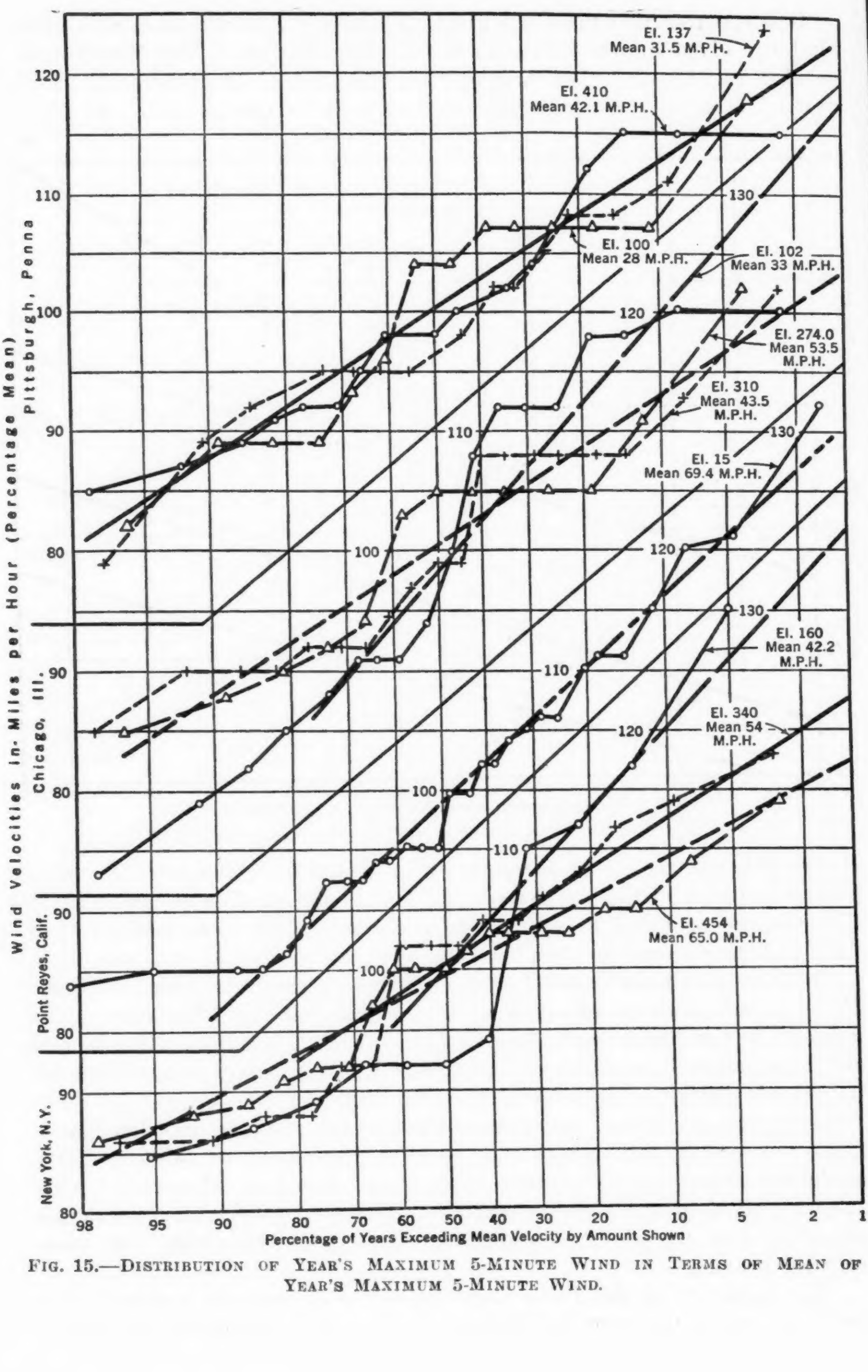


FIG. 15.—DISTRIBUTION OF YEAR'S MAXIMUM 5-MINUTE WIND IN TERMS OF MEAN OF YEAR'S MAXIMUM 5-MINUTE WIND.

time record tends to a uniform frequency curve. As there is no flattening of the curve at the top it appears probable that a record of 100 years would show still higher velocities.

A fact of interest is that the highest wind at Point Reyes was in 1895 and for New York probably in 1888 (allowing for the lower elevation of the anemometer), periods 37 and 44 years ago; this is important. The writer feels that there is a great tendency to forget the storms and disasters of a previous generation. After the Tay Bridge disaster in England (1879), 56 lb. per sq. ft. was assumed for a wind load. It has been continuously cut down since. Do engineers need another disaster to remind them that a one-year-in-a-hundred wind only occurs once in 100 years?

If a modern monumental building is assumed to have a life of 100 years, a 1% wind has an even chance of occurring during that time, and a 0.1%

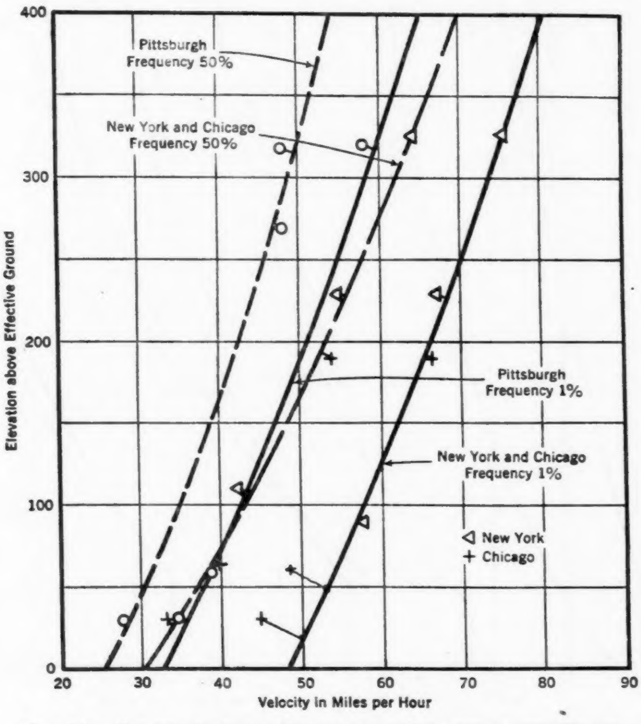


FIG. 16.—CURVES SHOWING INCREASE IN VELOCITY ABOVE EFFECTIVE GROUND (DATA TAKEN FROM FREQUENCY CURVES OF FIG. 15).

wind one chance in ten. Considering the contingent risks involved in the failure of a building, the additional 1% cost required to provide additional safety would seem a good insurance risk and the 0.1% wind would be in line with the safety factors used in the spillways of dams. Were the writer

responsible he would be content with nothing less. However, for the remainder of the discussion he uses the 1% wind, as being one which probably will occur within the life of a building.

In Fig. 16, the mean (50%) and the 1% winds taken from Fig. 17 have been plotted against "effective" ground in order to bring out the laws of increase of velocity with height above ground. What is desired is the law

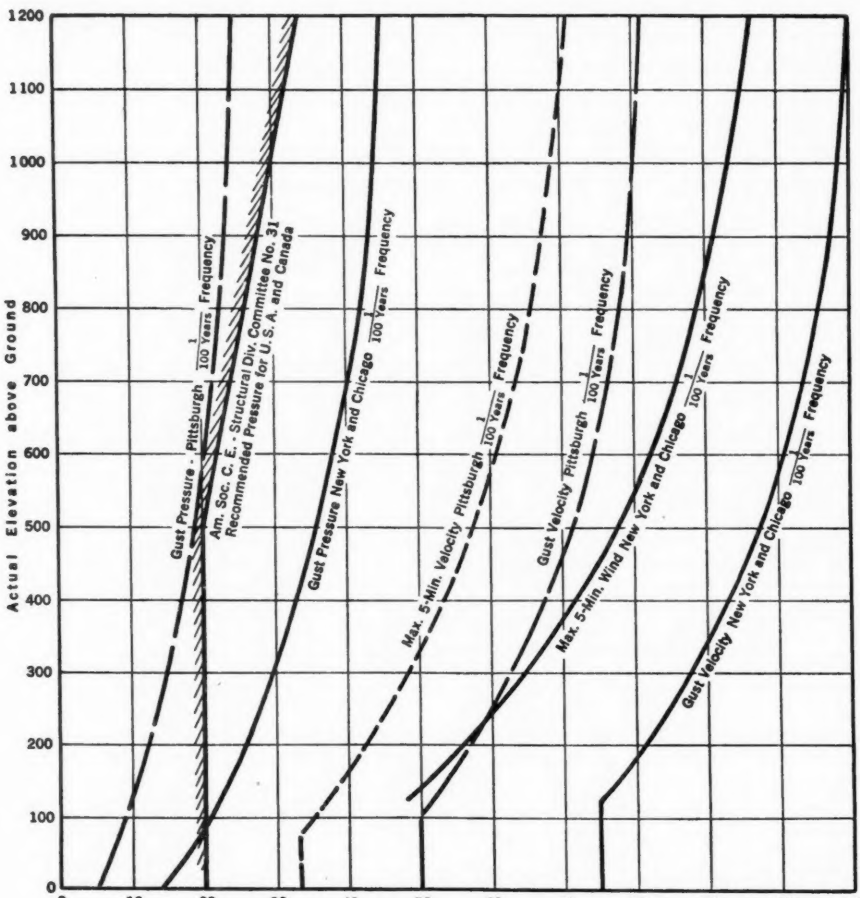


FIG. 17.—EXPECTED WIND PRESSURES AND VELOCITIES AT NEW YORK, N. Y., CHICAGO, ILL., AND PITTSBURGH, PA.

of increase in unobstructed country. In a city the writer feels that what he calls the "effective" ground is approximately three-quarters of the height of the average buildings over a $\frac{1}{4}$-mile area. Below this height the velocities will follow no regular law, due to obstructions of buildings and to gusts and eddies sweeping through the streets. The "effective" height would be the ground in open country or the tops of the trees over woods. The effective

height probably has risen gradually as cities have grown. The use of this conception may be open to criticism, but the corrections it involves have little effect in the resultant curves as applied to the cities from which the data are taken. In Table 2 the estimated wind velocities at "effective" ground have been taken from the curves of Fig. 16. At present, the effective ground height of New York has been estimated at 125 ft. The values used for all stations are shown in Table 2.

TABLE 2.—TABULATION OF DATA

Station (1)	Years of record (2)	Mean of yearly maximum 5-minute winds, in miles per hour. (3)	Elevation of anemometer, in feet. (4)	Effective ground elevation at period of record, in feet. (5)	Elevation of anemometer above effective ground, in feet. (6)	$\frac{50}{100}$ wind, in miles per hour. (7)	$\frac{1}{100}$ wind, in miles per hour. (8)	Percentage of $\frac{50}{100}$ wind in relation to that existing at "effective" ground. (9)	Percentage of $\frac{1}{100}$ wind in relation to that at "effective" ground. (10)
PITTSBURGH, PA.: Prevailing high wind, west.	....	.....	....	0	0	*25.5	*31.0	1.0	1.0
	1873– 86	28	100	70	30	27.2	34.5	1.06	1.12
	1887–1903	31.5	137	75	62	32.0	38.7	1.26	1.24
	1904– 10	48.0	352	80	272	48.0	....	1.86	......
	1911– 29	47.1	410	95	315	47.5	58.0	1.85	1.88
CHICAGO, ILL.: Prevailing high wind, southwest.	....	.....	....	0	.....	*31.0	*48.0	1.0	1.0
	1872– 89	33.0	102	70	32	32.6	45.0	1.06	0.95
	1892–1904	53.5	274	80	194	54.0	66.0	1.74	1.38
	1906– 24	43.5	310	95	215	44.0	54.0	......	......
NEW YORK, N. Y.: Prevailing high wind, northwest	....	.....	....	0	.....	*31.0	*48.0	1.0	1.0
	1872– 75	41.0	13	50	37	41.0	....	1.32	......
	1876– 86	42.7	160	70	90	41.5	58.5	1.33	1.22
	1887– 94	42.5	200	90	110	42.5	....	1.37	......
	1895–1910	54.0	340	110	230	55.0	67.0	1.77	1.40
	1911– 29	65.0	454	125	329	64.5	76.0	2.08	1.58
POINT REYES, CALIF.: Prevailing high wind, northwest	1892–1926	69.4	15	15	.....	69.0	90.0	......	......

* Estimated wind value at effective ground.

The results of the plotting are remarkably consistent considering the varied nature of the data. In all cases, except Chicago at Elevation 310, which disparity has already been explained, there is an increase in velocity with height, and the 50% and 1% winds show a consistent relationship to one another. It is to be noted that since the slopes of the velocity-elevation curves are about the same, at higher velocities there is a slightly smaller percentage of velocity increase with height. This is to be expected, due to the effect of great turbulence at the ground line equalizing the velocities at higher elevations.

Formula for Increase of Velocity with Height.—The general laws governing this increase in velocity and a formula expressing it are discussed by Lamb.[31] The treatment is involved, but the writer believes it would repay the study of some of the mathematicians of the Society. The writer has

[13] "Hydro-Dynamics," by Lamb, Fifth Edition, p. 655.

fitted what he hopes is an approximation of Lamb's equations to the experimental points and the resultant equation is:

$$V = 1.17\, V_g\, (1.854 - e^{-0.002\,(h-h)_g}) \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which, V = velocity, in miles per hour, at any height, h; V_g = velocity, in miles per hour, at "effective ground;" h = height above surface, in feet; h_g=height above surface of "effective ground" (about three-fourths the height of average buildings); and $d = h - h_g$. For New York, $V_g = 48$ and $h_g = 125$; and, for Pittsburgh, $V_g = 35$ and $h_g = 95$. These curves have been plotted in Fig. 17. Chicago's curve would be approximately the same as that for New York.

Since these curves differ from others proposed, it is desired to set out clearly the data on which they are based and the assumptions used in deriving them:

(1) The data which are a combined total of 150 years of observation, consist of the highest 5-min. wind of each year found at the stations of Chicago, Pittsburgh, and New York. At each of these stations at different periods the anemometers were located at not less than three different elevations for at least a 10-year period each. The yearly maximum velocities ranged from 30 to 74 miles per hour, and the anemometers were located as high as 454 ft. above ground, making the data unique, as compared with those used by any other experimenter, for the purposes of a structural engineer.

(2) In treating the data the assumption is made that the ratio of a wind of a given frequency at one elevation, to that of the same frequency at another elevation, is the same as if simultaneous measurements of a single wind were made. For extreme velocities it is believed this assumption is true.

(3) In extending the curves from Elevation 454 to Elevation 1 200, use is made of theory; of the statements of Dines and Marvin that the wind doubled at from 1 000 to 1 500 ft. above the ground; of analogy with water; of the shapes of the curves; and of the known velocities of the gradient winds. Gradient winds of 150 miles per hour occur yearly (storm of March, 1932, over New England). In the Florida hurricane, the value was from 300 to 400 miles per hour.

The writer feels that these data and similar records available in the Weather Bureau furnish the best field for determining the increase of wind velocity, which can become available for many, many years. Local knowledge plus a study of the gradient winds existing at the time of the records, and the use of more stations, would improve on the writer's results, but he believes his curves substantially represent the facts. The installation of pressure-recording apparatus on such buildings as the Empire State Building will provide useful data in determining pressure coefficients and unit stresses, but it will be many years before enough winds can be measured to compare with the value of existing records, as far as frequency distribution and the general laws of increase of velocity with height are concerned. At this time,

the writer would like to withdraw his previous curve on wind increase with height from technical circulation.[32] It represents the average results of winds up to 37 miles per hour, but is not to be compared in value with the present study.

Gust Velocities.—The Sub-Committee estimated gusts as being uniformly 25% greater than the 5-min. wind. General considerations indicate that the percentage increase of velocity of gusts and their variability are greater

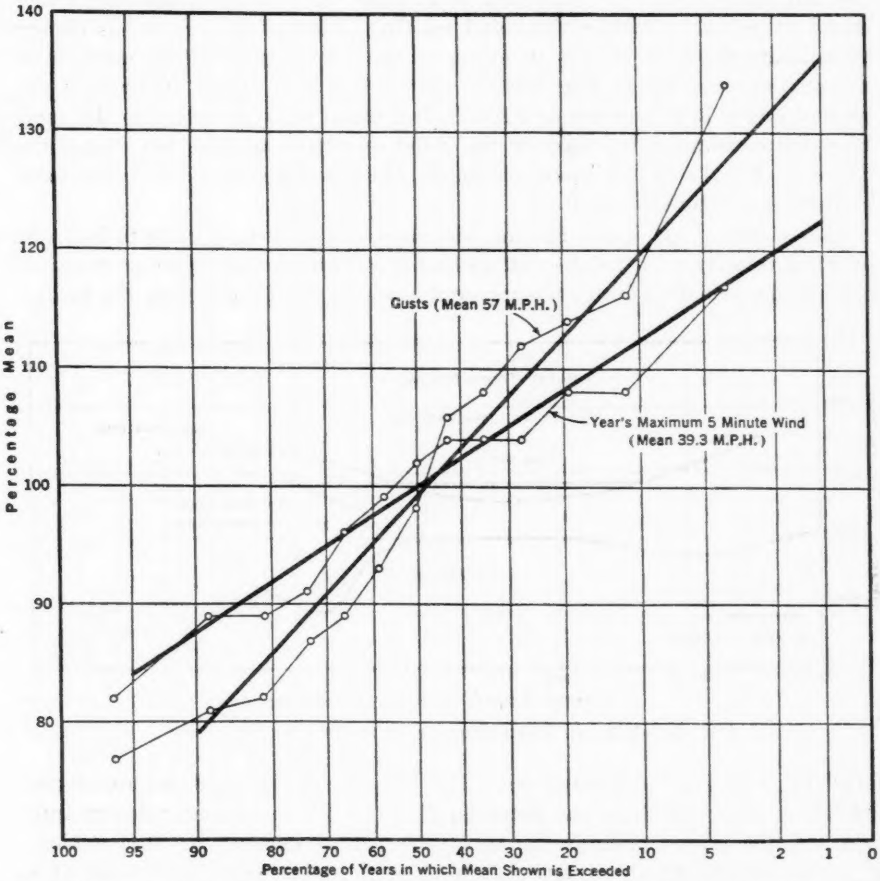


FIG. 18.—COMPARISON OF FREQUENCY RELATIONSHIPS BETWEEN GUSTS AND MAXIMUM YEARLY 5-MINUTE WINDS AT CHICAGO UNIVERSITY (STATION ELEVATION 131; "EFFECTIVE" GROUND, 70 FEET).

at low velocities and low elevations than for higher elevations and higher velocities. A given pressure change will produce a greater change in a low-velocity wind than in a high one. In Fig. 18 is presented a frequency comparison of the gusts which occurred at the time of the maximum yearly 5-min. wind at the Weather Bureau Station of the University of Chicago,

[32] "Wind Pressure and Design of Radio Towers," *The Electrician*, July 1, 1921.

131 ft. above the ground and analyzed for a 13-year period. The gusts averaged 45% greater than these maximum yearly winds. As might be expected the gusts are more variable than the 5-min. winds so that the one-year-in-a-hundred gust is 60% greater than the one-year-in-a-hundred, 5-min. wind. The writer feels that this value substantially represents the facts near the ground and such a gust, as Stanton showed[33] with a knowledge of the contrary previous experiments, will occasionally act across the entire width of the building. What the condition is at 1 200 ft. remains unknown. Influenced by general considerations and existing practice the writer has chosen to estimate the gust as 13% in excess of the 5-min. wind. This estimate is not on the conservative side, but the writer feels that a 50% increase at the ground and a 13% increase at 1 200 ft. is a closer approximation to the facts than the constant value used by the Sub-Committee, and he has drawn the curves in Fig. 17 on this basis. At 500 ft., the percentage increase is the same as that of the Sub-Committee.

Resistance Coefficient.—Having presented new information as to increase of wind velocity with height and frequency of winds, the writer proposes to convert the velocities into pressures and compare the results with the recom-

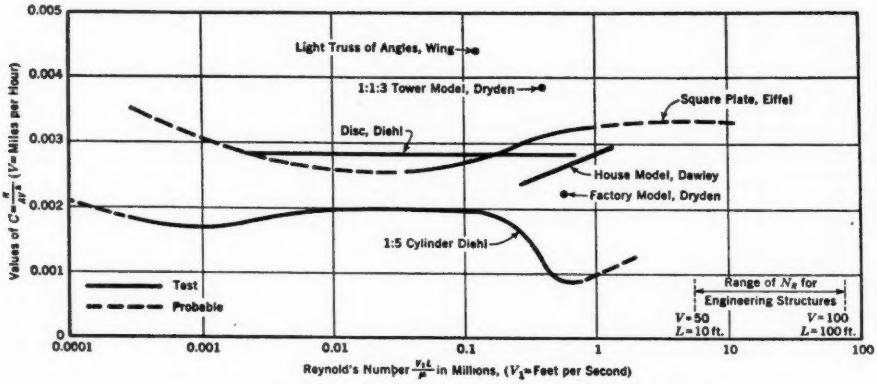


FIG. 19.—TYPICAL WIND RESISTANCE COEFFICIENT CURVES.

mendations of the Sub-Committee. The Sub-Committee used the resistance coefficient, $C = 0.0033$, in the formula, $P = C\,A\,V^2$, to convert velocity into pressure, the reason given being that it rested on careful experimentation. However, the experiments referred to were on the coefficient of resistance of a flat plate and have no known application to buildings. Since to convert velocity into pressure some coefficient, determined by model experiment, must be used, it would seem essential to base experiments on an approximate model of the structures in question, and the writer believes the variable coefficients proposed by Dryden and Hill[34] the best available. However, since the Sub-Committee wishes to use a fixed coefficient, Fig. 19 is submitted. This shows

[33] "Measurement of the Pressure of Wind on Structures," by Stanton, *Minutes of Proceedings*, Inst. C. E., 1924.

[34] "Wind Pressures on Structures," Dryden and Hill, *Scientific Paper 523*, Bureau of Standards.

resistance coefficient curves and experimental points for various shapes plotted against Reynolds' number. Let R = resistance, in pounds; P = resistance, in pounds per square foot; L = diameter or width, in feet; V_1 = velocity, in feet per second; V = velocity, in miles per hour; W = pounds per cubic foot; P = density $= \frac{W}{g}$; H = height, in feet; A = area, in feet $= H\ L$; ν = kinematic viscosity = density viscosity (feet per second units); C = dimensionless coefficient; and N_R = Reynolds' number $= \frac{V_1 L}{\nu}$.

It has been found by inumerable experiments that, if the resistance—measured in terms of velocity head per unit area, that any similar shape of whatever size offers to the flow of any fluid (such as gas, oil, or water)—be plotted against Reynolds' number, a smooth curve results.

Briefly, ϕ is a function of the shape and N_R in the formula,

$$\frac{R}{A W} = C_1 \frac{V_1^2}{2 g} \quad \text{.........................(7)}$$

or, if similar shapes are used throughout a series of experiments, $C_1 = (F)\ \frac{(V L)}{\nu}$ only, and all experimental determinations of C_1 — whether made with a big model of an area of 100 sq. ft., or a small model of an area of 0.1 sq. ft.; whether tested at high velocities or at low velocities; or whether the test is made in oil, water, or air — will fall on the same curve, within experimental limits (subject to the usual engineering exceptions). For the purposes of this discussion in Fig. 18, C in the formula, $P = C\ A\ V^2$, has been substituted for C_1 ($C = 0.00256\ C_1$). A value of $C = 0.00256$ means a unit resistance equal to the velocity head.

Scale Effect.—It will be noticed that the curves are quite irregular, varying both with shape and N_R; yet independent experimenters working with different models at different velocities will check them with reasonable accuracy. The variation of the curves with N_R explains the results that earlier experimenters found in the values for pressure of wind on a square plate. What was called scale effect was really the expression of a change in N_R. In this sense, scale effect does not exist. Providing the C-N_R curve has been adequately defined by experiment in the range of N_R, desired for the prototype, no uncertainty in the value of the coefficient exists. Unfortunately, in the range of N_R required for practical engineering structures, no suitable experiments have been made. Luckily, shapes differ greatly in their characteristic curves. It will be noted that the curve for a 1:3 cylinder varies widely with N_R and this is characteristic of cylinders and ovals. On the contrary, the curves for a disk or a plate are comparatively uniform over a wide range. Dryden[35] states that this is likely to be characteristic of the curves of buildings. Consequently, it is possible to extrapolate with reasonable accuracy from existing points obtained by a model test to the value of N_R which the prototype will have.

[35] "Wind Pressure on a Model of Mill Building," by Dryden and Hill, *Research Paper No. 301*, Bureau of Standards.

One other type of scale effect needs to be noted and this is the effect of surface. The N_R function depends on shapes being similar not only in form but in surface. Experiments on the resistance to flow of pipes plotted against N_R indicate that as size is increased, holding N_R constant by diminishing velocity to obtain the same values of C, the relative roughness also must be increased to obtain the same loss. Thus, if a 1:200 scale model were made of planed wood it might be expected to represent in surface a prototype of rough masonry. If, however, the model does not reproduce windows, etc., to scale, it is probable that the resistance of the prototype would be under-estimated. For such shapes as buildings, however, the effect of surface friction is small, and any correction is likely to be of the order of 3 or 4 per cent.

Now consider the relationship of N_R to models. It has been shown that for certain shapes it is not necessary to test at full N_R to obtain reasonable results. Consider the action of a 100 by 100 by 1 000-ft. building in a 100-mile wind. Its N_R will be $100 \times \dfrac{5\,280}{3\,600} \times \dfrac{100}{15.6 \times 10^{-5}} = 95\,000\,000$. If a model is tested in air at $N_R = 475\,000$, a good estimate of the full scale value of C can be made. A model to a 1:200 scale would have this value, and the model would be 6 by 6 by 60 in. If this same model is tested in water at one-fifteenth the velocity in air (10 ft per sec.), the same value of the resistance coefficient will be obtained since the kinematic viscosity of water is about one-fifteenth that of air. The writer believes that in estimating the effects of wind on buildings of a modern city an excellent mental picture of events can be had by thinking of what would happen to a model city to 1:200 scale tested in a stream of water 10 ft. deep, flowing at a velocity of 10 ft. per sec. The ordinary buildings would be represented by varying parallelopipeds, 12 to 24 in. high, while the tower buildings would be 6 by 6 by 60-in. posts spaced irregularly at 5-ft. centers. The turbulences and phenomena which would occur in this hydraulic model will be accurately reproduced in the air flowing over the actual city at the same N_R. Skeptics are referred to the beautiful illustrations of such effects made visible in air and water by Bairstow.[36] Of course, the model being considered does not reproduce the effect of gusts originating above the elevation of the gradient wind, but the effects of such gusts should be similar.

In the first place in such a stream as is being considered, with a rough bottom there would be a large increase in velocity from the bottom upward. This is characteristic of the flow of water in all channels, the maximum velocity commonly being found to be 1.7 to 2.0 times the bottom velocity.[37] The rougher the bottom the higher the point at which maximum velocity is found. This percentage of increase of velocity is a rough check on the values used for the ratio of the gradient to the ground wind. Furthermore, this characteristic increase in velocity holds at high as well as at low velocities. Hence, it seems probable that in the case of an air stream this same

[36] "Applied Aerodynamics," by Bairstow, p. 378.
[37] "Hydraulics and Its Applications,' by Gibson.

increase of velocity will be found at high velocities also. Moreover, in the case of the air stream, there is no retarding effect of a contact surface such as a water stream meets at its surface to limit such increase.

If attention is turned to what is happening at the base of the imaginary model currents of high velocity would be found sweeping through the model streets, forming boils and eddies which would occasionally reach the surface with diminished intensity and greatly increased areas. It seems reasonable to suppose that their size would be much greater than 6 in., the size of a model tower. Occasionally, they would hit a model building and the effect of impact would be decidedly noticeable. From the down-stream edge of each model building a series of vortices would be noted, and an occasional building would be noticed oscillating, much as the moving branch of an overhanging tree at the edge of a stream, so often seen along a river. The characteristics of this oscillation and of the vortices have been analyzed for certain shapes. For cylinders in air it has been found that vortices form alternately from either side with a period equal to $5 - 7 \frac{L}{V_1}$.[38] When this period coincides with the natural period of the obstruction, oscillation takes place. If a square shape has a period of the same order of magnitude as a cylinder, for a 100-ft. square building and a velocity of 100 miles per hour, the eddy period would be of the order of 3 to 5 sec. Since the natural period of a high building is about the same, synchronism may easily be possible, aside from the effects of gusts.

Thus, the consideration of a hydraulic model, with its phenomena correlated to air by N_R, has led to a confirmation of the experimentally found increase of wind velocity with height, and has given presumptive evidence that gusts will be distributed over fairly wide areas, that little shielding of the upper parts of the buildings will take place, and that impact and synchronous vibration are quite probable. These factors all have a bearing on the decision of what coefficient to use in converting velocity into pressure. Dryden and Hill[34] have proposed the factor, 0.004. Considering that tall buildings spring from a wide base the writer has taken 0.00375 and on account of past practice has reluctantly decided to leave the impact, etc., effects to be taken care of in the unit stress.

Discussion of Proposed Wind Loadings.—Fig. 17 shows the writer's estimate of the pressure expected during the life of an unshielded building at New York, Chicago, and Pittsburgh, as compared with the recommendations of the Sub-Committee. For a 1 000-ft. building, at a point 500 ft. above the ground, at least once during the life of the building the expected load would be about 150% of that provided by the Sub-Committee's recommendations, and in about one year in two the recommended loading would be reached.

Under "Prescribed Wind Force," the Sub-Committee states that the wind force prescription "should be adequate for any part of the United States or Canada." No doubt the intended meaning does not imply that the unit stresses will always be less than 24 000 lb. per sq. in., but that the building

[38] "Applied Aerodynamics," by Thompson, p. 30.

will stand with somewhat the same success as those in Miami, where some designed to the Sub-Committee's specifications survived and others failed.[39] It is true that in examination of the partly destroyed structure details were found which could have been improved upon, but the writer doubts whether the majority of buildings of similar character elsewhere would be found without similarly weak details upon a critical examination by experts.

It is instructive to consider the Miami structures a little further since the Sub-Committee cites them in support of its statement. The Committee on the Florida Hurricane estimated the load on the complete structures as of the order of 50 lb. per sq. ft., but stated of the buildings that partly failed, that it was probable that a 20-lb. wind would cause stress in excess of 24 000 lb. per sq. in. in some members. If it be assumed that the dead and live load stresses were of the order of 16 000 lb. per sq. in., then the 50-lb. wind load would have caused stresses of approximately 36 000 lb. On the other hand, if the connections were computed without dead load, on a 24 000-lb. base for a wind load of 20 lb., and a load of 50 lb. was applied, it would appear that they were stressed to 60 000 lb. It seems clear that the actual load must have been less than that assumed by the Committee, or considerable loads were carried by the masonry. In either case, these data would appear to be poor grounds for assuming that a 20-lb. wind load is adequate for a structure 500 ft. high, in Miami. The writer's curves indicate that a 30% increase in load would have occurred at 500 ft. over that at 250 ft. (the highest building at the time) and it may well be doubted if many 500-ft. structures would have survived.

To check the Sub-Committee's statement for other localities the writer has roughly computed the expected wind loads at 500 ft. in terms of the Sub-Committee's recommended 20 lb., for several places in the United States, basing his estimate on the mean yearly 5-min. wind, corrected for elevation. The following can be expected to be only a very rough classification:

Committee's Recommendation[40].	1.0	Chicago. Ill.	1.75
Los Angeles, Calif.	0.6	New York, N. Y.	1.75
Pittsburgh, Pa.	1.0	Dallas, Tex.	2.0
San Francisco, Calif.	1.1	Mobile, Ala.	2.3
Seattle, Wash.	1.2	Point Reyes, Calif.	3.5
St. Paul, Minn.	1.5		

It is interesting to note, considering winds and resistance coefficients together, that whereas, in Los Angeles, an aeroplane hanger, 100 ft. high, could be designed for a wind force of $P = 0.002\ (50)^2 = 5$ lb. per sq. ft., at Point Reyes, a 100-ft. electric sign, composed of an angle frame, would require a design load of $P = 0.0044\ (125)^2 = 70$ lb. per sq. ft. Illustrative of the inconsistencies of present practice is a recent 400-ft. building constructed at Pittsburgh for a wind load of 30 lb. per sq. ft.[41] whereas, in New York, 20

[39] Final Report of the Committee of the Structural Division on Florida Hurricane, *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1118.

[40] 20 lb. per sq. ft., at 500 ft.

[41] *Engineering News-Record*, April 7, 1932, p. 511.

lb. was thought safe up to 1 000 ft.[42] The writer's figures would have called for about 15 lb. at Pittsburgh and 40 lb. at New York, averaged over the entire building.

Before leaving the curves of pressure it should be noted that they are devised for a tall, unshielded tower type of building. The writer regrets that the Sub-Committee has lumped all buildings under one specification. The ordinary building would have considerable less pressure on it, due to the changed resistance coefficient (Fig. 18). If for simplification it is desirable to have only one specification for all buildings, the writer would suggest for New York the following, allowing for shielding in the lower 200 ft.:

Wind load, in pounds per foot	Building heights, in feet
10	0 to 100 ft.
20	100 to 200 ft.
30	200 to 400 ft.

For all buildings higher than 400 ft. add 3 lb. to the wind load for each additional 100 ft. Finally, based on the data examined, the writer would estimate the accuracy of his curve of pressure at ± 40 per cent. The Sub-Committee's curve might represent the true conditions for New York, but the writer feels that it is the minimum possible for that district and that there are many places in the country where it must be greatly exceeded.

Unit Stresses.—The Sub-Committee has followed past practice in allowing a higher unit stress for combined dead, live, and wind loads than for dead and live loads alone. This has all the prestige of past custom, but the custom does not bear critical examination. This practice undoubtedly is derived from older bridge specifications. For bridges, this is a logical provision. The live loads on the usual bridge are large, the wind loads relatively small in most members, and the probability of maximum live loads being combined with maximum wind loads is remote if not impossible since a train traveling with empty box cars will turn over in a 100-mile wind. For a building, on the contrary, most of the live load is always applied, and the magnitude of the wind load stresses, in parts at least, compares with those due to live loads. In addition, no allowance is made for impact or vibration. Moreover, the loads are repeated millions of times due to the natural variability of the wind, and in some members the stresses become reversed in sign. It is stated sometimes that increased unit stresses should be permitted for wind on account of their occasional character, but the frequency curves of the writer show that about two-thirds of the maximum values occur every other year, and it is probable that live loads reach their maximums no oftener. The writer believes that the sooner wind load is treated as a normal live load the better for structural engineers. However, he does not believe that the permitted stress due to wind should be much reduced, but rather that the other unit stresses should be raised, since the fact that buildings are standing up under high wind loads when the unit stresses in some members must often be of the order of 30 000 lb. per sq. in., is indicative of reserve strength.

[42] *Engineering News-Record,* January, 1931, p. 199.

Under the Sub-Committee's specification such reserve strength is very unevenly distributed, both as between members with and without wind and as between the various parts that make up the wind frame. If a member has a dead and live load stress of 18 000 lb. per sq. in., and a wind stress of 6 000 lb. per sq. in., the wind load may be doubled and the total unit stress is 30 000. In addition, in both columns and floor-beams, recent tests have shown an additional strength where columns are encased in concrete[43] and concrete floors are used,[44] of at least 10 to 20 per cent. The Society's Special Committee on Stresses in Structural Steel in 1925 recommended a 20 000 lb. unit base for steel stresses.[45] With proper provision governing the construction of the fire-proofing and floors and assuming that maximum wind loads are specified, the unit stresses for beams and columns might well be put on a 22 000-lb. unit base. As a matter of fact, as the writer has just shown, the base on which many structural members are working to-day must be in excess of this. However, according to the proposed specifications, all members are not in such a fortunate position. The uplift provision of the Sub-Committee of a tensile stress of two-thirds the dead load due to wind will allow a 50% margin over its specified wind load with an indeterminate additional factor due to any live load the column carries. According to the writer's curves this margin will all be needed in New York with no allowance for contingencies.

Finally, the writer knows of no justification, except expediency, for the Sub-Committee's specification of a 24 000-lb. unit stress base for bolts and rivets, especially taking into account its commendable reduction of pure wind stress to an 18 000-lb. value. Every consideration of test, experience, and theory indicates that the joint is the weak point in a rigid frame. Professors Wilson and Moore[46] showed that at about a 16 000-lb. base, permanent slip in the connections was beginning to be appreciable. In the Florida hurricane, original failure was probably due to weak joints.[39] U. T. Berg, Assoc. M. Am. Soc. C. E., has brought out[47] the effect of neglect of dead and live loads on the stresses in the end connections of beams (a point which has worried many designers); and, finally, numerous codes specify the relative relationships that must exist between stresses in rivets and bolts and those in the main members in order to have equal factors of safety. In addition, considerations of rigidity almost compel a lower unit stress in the connection since under wind loads the stress reverses in sign, and an inelastic slip in one direction is followed by inelastic slip in the other. The writer is well aware of the difficulty of providing connections at lower unit stresses, but the difficulty and expense can be no justification where security is involved. With modern welding, or a use of reinforced concrete, a satisfactory joint

[43] *Engineering News-Record*, September, 1929, p. 372, and October, 1931, "Brick Encased Columns."

[44] "Steel I-Beams Haunched in Concrete," by Gillespie, *Bulletin No. 5*, Univ. of Toronto.

[45] *Proceedings*, Am. Soc. C. E., March, 1925, Papers and Discussions, p. 392.

[46] "The Rigidity of Riveted Joints," by Wilson and Moore, *Bulletin No. 104*, Eng. Experiment Station, Univ. of Illinois.

[47] "Wind-Bracing Connection Efficiency," *Proceedings*, Am. Soc. C. E., January, 1932, p. 3.

can be evolved. The writer feels that the decrease of the rivet stresses in joints and the raising of the stresses for dead and live loads will produce a safer, cheaper, and better balanced design.

Negative Pressures.—Finally, the Sub-Committee includes no definite recommendation to take care of negative wind pressures. Such pressures and the wide areas over which they act, sometimes including three-fourths of the building,[34] are among the most characteristic phenomena of wind loading. Although commonly known this characteristic has been often overlooked by many, including the writer. Neglect of it has been more characteristic of wind failures than any other cause, and is responsible for much of the damage and loss of life in a wind storm, due to detached roofing and sidings being blown through the streets. The writer believes the specification should state that all siding, roofing, and trusses shall be designed for both external and internal moving loads equal to the amount of the wind specification.

Conclusion.—Although the writer does not agree to some of the provisions of the Sub-Committee's report, it is certain that it has been of great value in covering both design and loading in a neglected field. He will not be disappointed if the combined experience of the Sub-Committee rejects many of his opinions. He feels, however, that his 5-min. wind velocity and frequency curves are worth serious study and that it is probable that greater loads than those specified by the Sub-Committee are fairly common in exposed locations in the United States and Canada.

A. H. Finlay,[48] Assoc. M. Am. Soc. C. E. (by letter).[48a]—The problem of proportioning the steel frames of buildings so as to resist wind forces is, like Gaul of old, "divided into three parts." Obvious though these divisions are they may bear repeating: First, the intensity of the wind pressure must be decided upon; second, the share of the total wind load resisted by each bent in the structure must be decided; and, third, the further sub-division of the load carried by each bent among the individual members must be determined. The final result, as expressed in design sections, merely reflects the assumptions made along the way.

With regard to the Sub-Committee's specified wind pressure the writer is unable to express an opinion. Quantitative information on this baffling, yet all-important, phenomenon is far from being plentiful. A psychological aspect, of course, exists. If the prescribed wind pressure is high (and by high the writer means any value greater than 20 lb. per sq. ft.), the resulting shears, even in buildings of moderate height, may prove excessive. As a result embarrassing over-stress may be found to exist in certain members and connections. The designer, in such a case, remembers the high unit pressure used. His mind dwells upon the rarity of occurrence of the winds represented by that pressure, and the over-stress is conveniently "forgotten." Such experiences are not uncommon. The writer feels that the prescribed unit pressure should be kept low and the resulting stresses faithfully taken care of as stresses that are reasonably apt to occur. The resulting structure will not be one whit weaker or stronger, but it will at least have been rationally designed.

[48] Asst. Prof., Civ. Eng., Univ. of British Columbia, Vancouver, B. C., Canada.
[48a] Received by the Secretary April 23, 1932.

The old adage, "familiarity breeds contempt," is nowhere more strongly in evidence than in connection with wind-bracing design. Twenty years ago (it was only in 1912), the design of a 25-story building was an achievement of note. Articles appeared in engineering journals describing the elaborate means by which such buildings were enabled to resist wind forces. Diagonal bracing, knee-braces, and deep gusset-plate connections between girders and columns were the expedients used in such structures.[40] Contrast this with the totally inadequate top and bottom clip angles which to-day would in all probability comprise the "wind-bracing" accorded such a structure.

About 1928 the writer reported upon the wind-bracing (or more properly upon the lack of it) in a 24-story office building of somewhat odd shape. In this building as designed he found absolutely no provision for wind-bracing of any kind. A reinforced concrete ribbed floor was used and the beams that framed into the columns parallel to the ribs were mere 8-in. **I**-struts. Indeed the frame as originally designed possessed so little rigidity that even its erection, without special precautions, would have been hazardous. The writer does not believe this case to be typical, but it serves to illustrate the aptness of the old adage.

The majority of office workers, hotel guests, and apartment dwellers is not sheltered in the Empire State Buildings of the world any more than the bulk of water-borne cargoes is carried in the *Majestics* and the *Leviathans*. It is sheltered in buildings of moderate height. It is in the design of these buildings that human carelessness must be guarded against. The daring proportions of modern tower-like buildings are sufficient assurance that lateral forces will be carefully considered.

Having assumed a unit pressure, the second phase of the problem, about which little is heard, as a rule, presents, in the writer's opinion, the most difficult part of all. By allotting each bent in the building a share of the wind load the design is established in its most important particular. That the allotted share is as nearly correct as possible is obviously of the greatest importance. No results beyond this point can be any better than the assumption made at this stage, regardless of the method by which, at a later stage, the individual column shears and girder moments are determined.

The statement is often made that the rigid floor-slabs of a building cause the tops of all columns to deflect equally. The writer has never seen this questioned and yet it would appear open to question in all those cases in which the center of lateral resistance of a building at any level does not coincide with the line of action of the resultant wind force on the structure above that level. It will be realized that the center of lateral resistance in a building may be on a different line at different levels since it depends primarily upon the number and depth of the wind bents and rigid walls available at the level in question. The line of action of the resultant of the wind forces on the structure above any level will change from level to level with changes in the exposed area. It is thus apparent that either of these factors may change independently of the other. The laws of statics require, among other things, that the moment of the column shears in any story combined

[40] *The Engineering Record*, December, 1911.

with the torsional moments which may exist in the columns of the same story be in equilibrium about the line of action of the resultant of the wind forces on the structure above such a story. The building will twist until such a condition obtains. It will be realized from the foregoing that wind bents of identical construction may resist quite different shares of the total wind load, dependent upon their position in the structure and the angle of twist involved. The problem is further complicated by the fact that the division of the total wind load between bents will change from floor to floor.

It is not the writer's intention to elaborate this aspect further and it is beyond his present ability to suggest a means whereby the correct loadings for each bent could be established. He would be most interested to hear the opinion of the Sub-Committee on this important point.

Having assumed some arbitrary loading for each bent there remains to determine the resulting bending moments and shears in the individual members. If, at this point, in the face of the assumptions which have gone before, it is still desired to determine them by any of the so-called "exact" methods the proceeding, even when aided by Professor Cross' method of moment distribution, is a tedious one. Several methods of applying moment distribution to this problem, in which joint movements play so important a part, are available, of which the one adopted by the Sub-Committee is probably as direct as any. Professor Cross some years ago outlined a method in which rotations at each joint are dealt with, rather than moments. It possesses the advantage that while there are as many unknown moments at a joint as there are members there is only one rotation. The various joint rotations are solved for by trial. It is particularly expeditious when applied to the solution of secondary stresses in trusses. Its application to the present problem is complicated, just as is moment distribution, by the need of constantly correcting for joint movements. Its virtue lies in the smaller number of quantities involved.

The writer feels that, in those cases in which the loads on each bent cannot be definitely foretold, any attempt at an "exact" analysis is not warranted, and that recourse might better be had to assumed column shears and points of contraflexure after giving more thought than is customary to the vital question of the distribution of the total wind load between bents. The analysis of wind stresses in buildings is, after all, a problem in space rather than in a plane.

As indicated by the Sub-Committee the matter of rigid details is of paramount importance in wind-bracing design. Slip in connections is objectionable from two standpoints. The values of the bending moments are modified. Since it is largely the relative rather than the absolute values of the slips which affect the bending moments the objection on this score is perhaps not so serious as it might be. The effect of slip upon deflection is, of course, very pronounced and because of this fact alone slip of connections must be reduced to a minimum. The writer feels that the top and bottom clip-angle type of connection is inadequate in this respect. While the tests to determine the rigidity of riveted joints reported by Professors Wilson and Moore[50] did not

[50] *Bulletin 104*, Eng. Experiment Station, Univ. of Illinois.

cover a combination of top and bottom clip angles with the standard web connection, each of these connections was tested separately and found wanting in rigidity. Any type of connection that depends for its rigidity upon the ability of a thin member to resist cross-bending must be unsatisfactory from the point of view of stiffness.

The **I**-beam or **H**-beam stub connection is, of course, very useful. Since such an arrangement ordinarily precludes the use of adequate web connection the question arises as to its strength and stiffness in shear. The writer would be interested to hear of tests of this type of joint.

The Sub-Committee is to be congratulated upon its endeavor to correct the impression, still strongly held in some quarters, that rivets in tension are unreliable. The exhaustive tests at the University of Toronto under Professor Young and at the University of Illinois under Professor Wilson surely should have laid this "ghost." In view of these tests might it not be advisable to specify working stresses for rivets in tension?

The Sub-Committee's recommendations regarding limiting stresses in members stressed by dead load, live load, and wind are eminently logical. The results of deflection computations, as applied to actual buildings, surely cannot be as reliable as the Sub-Committee's Conclusion (5) would indicate.

J. D. GEDO,[51] ESQ. (by letter).[51a]—In Section (2) of this report, the Sub-Committee states that the slope-deflection method, or any other of the so-called "exact" methods, is too laborious to be practicable without some form of abbreviation, and that this is likely to result in an objectionable degree of inaccuracy, or at least of uncertainty as to the accuracy of the results.

On the contrary, the writer wishes to state that it is easy to decide whether or not such results are correct. First, there are the static criteria that (1) the summation of the moments must equal zero at each joint; and (2) that the

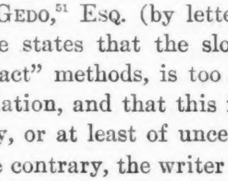


FIG. 20.

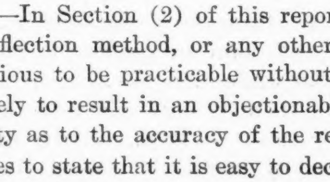


FIG. 21.

sum of the moments at the top and bottom of all the columns of a story is equal to the shear in the story multiplied by the story height.

In addition to these well-known static criteria, there are the elastic criteria. In the case of building bents, as in any other case, they can be deducted from the theorem of least work. The writer has demonstrated this elsewhere.[52] The elastic criteria in this case are, as follows: The rectangle in Fig. 20 may be

[51] Structural Designer, Alexander D. Crosett, Cons. Engr., New York, N. Y.
[51a] Received by the Secretary May 24, 1932.
[52] "Das Kräfteplan-Verfahren," Verlag Alfred Kröner, Leipzig, Germany.

considered as being a part of any bent; then the following three relations hold:

$$h_R\, M_{4B} - h_L\, M_{3B} = 2l_A\,(M_{3R} - M_{4L}) + l_B\,(M_{1R} - M_{2L}) \quad \ldots\ldots\ldots (8a)$$

$$3h_R\,(M_{2A} - M_{4B}) = -\,l_A\,(M_{3R} - 2M_{4L}) + l_B\,(M_{1R} - 2M_{2L}) \quad \ldots\ldots (8b)$$

and,

$$3h_L\,(M_{3B} - M_{1A}) = -\,l_A\,(2M_{3R} - M_{4L}) + l_B\,(2M_{1R} - M_{2L}) \quad \ldots (8c)$$

in which, the subscripts, A, B, L, and R, signify "Above," "Below," "Left," and "Right," respectively. For instance, M_{4B} is the moment immediately below Joint 4. The coefficients, h and l, are in some arbitrary ratio to $\frac{H}{I}$ and $\frac{L}{I}$, respectively.

In the rectangle, Fig. 21, this arbitrary ratio is selected as 200, so that,

$$h_R = h_L = \frac{200 \times 14}{5\,946} = 0.4709$$

and,

$$l_A = l_B = \frac{200 \times 22}{5\,641} = 0.7800$$

Fig. 21 is a part of the bent analyzed by Wilson and Maney,[53] who determined the following moments for this rectangle by the slope-deflection method: $M_{1A} = 90\,700$; $M_{1R} = 203\,000$; $M_{2A} = 182\,000$; $M_{2L} = 184\,000$; $M_{3B} = 100\,300$; $M_{3R} = 187\,100$; $M_{4B} = 187\,500$; and, $M_{4L} = 171\,700$. It is easy to see whether these values satisfy Equations (8*a*), (8*b*), and (8*c*); thus, for example: $0.4709\;(187\,500 - 100\,300) = 0.7800\;[2\;(187\,100 - 171\,700) + 203\,000 - 184\,000]$ (see Equation (8*a*), or, $41\,062 = 38\,844$.

As may be seen, the check is not quite satisfactory. If the writer's equations are correct, the discrepancy may be explained by the fact that the slope-deflection method is based on the assumption that the angular changes at a joint are all equal, or, in other words, that the joints are rigid.

Equation (8*a*) is almost perfectly satisfied with the following moments (derived from the theorem of least work): $M_{1A} = 90\,961$; $M_{1R} = 203\,347$; $M_{2A} = 182\,627$; $M_{2L} = 184\,040$; $M_{3B} = 100\,486$; $M_{3R} = 188\,674$; $M_{4B} = 187\,885$; and $M_{4L} = 171\,943$; then, $0.4709\;(187\,885 - 100\,486) = 0.7800\;[2\,(188\,674 - 171\,943) + 203\,347 - 184\,040]$, or $41\,156 = 41\,160$.

The lowest story of a bent is not a quadrangle. If the bases are fixed as in Fig. 22, then in every bay three elastic criteria may be written:

$$h_R\, M_{4B} - h_L\, M_{3B} = 2\,(h_R\, M_2 - h_L\, M_1) \quad \ldots\ldots\ldots\ldots\ldots (9a)$$

$$3h_R\,(M_2 - M_{4B}) = -\,l\,(M_{3R} - 2M_{4L}) \quad \ldots\ldots\ldots\ldots\ldots (9b)$$

and,

$$3h_L\,(M_{3B} - M_1) = -\,l\,(2M_{3R} - M_{4L}) \quad \ldots\ldots\ldots\ldots\ldots (9c)$$

[53] *Bulletin No. 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

These criteria may be tested on an example given[54] by G. E. Large, Assoc. M. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., who found the

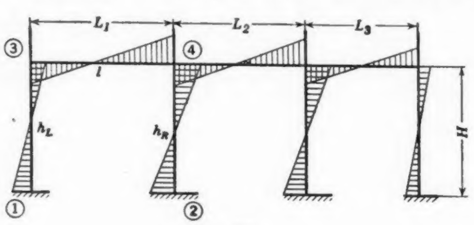


FIG. 22.

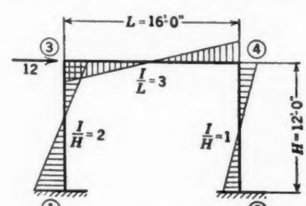


FIG. 23.

following moments in Fig. 23 by the "elastic theory": $M_1 = 49.72$; $M_2 = 28.71$; $M_{3B} = M_{3R} = 38.57$; and, $M_{4B} = M_{4L} = 27.00$.

The values, h and l, are selected as the reciprocal values of $\frac{I}{H}$ and $\frac{I}{L}$, and, therefore, $h_L = \frac{1}{2}$; $l = \frac{1}{3}$; and $h_R = 1$. Then (see Equations (9)),

$$1 \times 27.00 - \frac{1}{2} \times 38.57 = 2\,(28.71 - \frac{1}{2} \times 49.72)$$

$$3\,(28.71 - 27.00) = -\frac{1}{3}(38.57 - 2 \times 27.00)$$

and,

$$3 \times \frac{1}{2}(38.57 - 49.72) = -\frac{1}{3}\,(2 \times 38.57 - 27.00)$$

All these criteria are closely fulfilled. The insignificant discrepancy is due entirely to the fact that Professors Large and Morris (as any one would) stopped at the second decimal. Equations (9) would be perfectly fulfilled with the following values: $M_1 = \frac{348}{7} = 49.714286$; $M_2 = \frac{201}{7} = 28.714286$; $M_3 = \frac{270}{7} = 38.571429$; and, $M_4 = \frac{189}{7} = 27.000000$.

The writer could enumerate several other criteria covering the cases of offsets, hinged supports, and unequal bottom story heights; and, in addition, criteria that take into account the work of direct forces and shearing forces. He feels, however, that he has shown enough to bring out his point; namely, that there are definite equations to every statically indeterminate problem that are the criteria as to the accuracy of the results.

V. A. VANONI,[55] JUN. AM. SOC. C. E., and M. P. WHITE,[56] ESQ. (by letter).[56a]—A rather brief investigation made by the writers disclosed only one reference to the subject of secondary stresses in building bents due to column shortening under wind load. In "Wind Stresses in the Steel Frames of Office

[54] "The Moment Distribution Method of Structural Analysis Extended to Lateral Loads and Members of Variable Section," *Bulletin No. 66,* Ohio State Univ., pp. 5-6.
[55] Estimator, Eng. Dept., McClintic-Marshall Co., Los Angeles, Calif.
[56] Asst. in Civ. Eng., California Inst. of Technology, Pasadena, Calif.
[56a] Received by the Secretary May 24, 1932.

Buildings," Professors Wilson and Maney cover the subject in a paragraph.[57] The shortening of the first-story columns of the twenty-story bent is computed and the fixed-end moments in the first-story girders are determined. These moments are small compared with the original wind moments. However, it must be remembered that the effect of column shortening is cumulative from bottom to top, while the girder moments due to wind become smaller, so that the effect of column shortening should be much greater at the top than near the ground.

The Sub-Committee states that in the case of a high, narrow building secondary moments require investigation. In the relatively high and narrow Wilson-Maney bent, secondary moments are negligible. Apparently, there is another criterion. The importance of secondary moments depends upon the relative size of bays and the relative stiffness of column and girders. In the Wilson-Maney bent, both columns on one side of the center line have the same kind of stress under wind load and this stress (and, therefore, the shortening) is roughly proportional to the distance from the center line. Since all the fixed-end moments in the girders due to column shortening act in the same direction, they will all be reduced by the resulting side-sway and, in their final position (that is, after equilibrium is reached), the girders will be nearly straight. When alternate tension and compression occur in the columns, the girders are constrained and, therefore, may have large bending moments after equilibrium is reached. In such a case side-sway will increase certain girder moments and decrease others.

The Sub-Committee does not consider the possibility that the secondary moments as first obtained may require correction. For example, in general (not always), the secondary moments in the girders will be opposite the primary moments. If the secondaries are large, say 50% of the primaries, the resultant moment will be one-half the original; but the secondaries will also produce column shortening which will cause more secondary moments, ordinarily

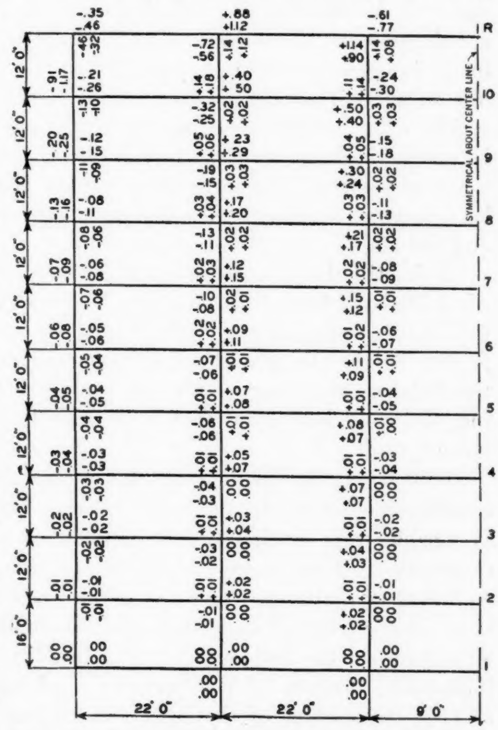


Fig. 24.—Ratios of First Secondary Moments and Corrected Secondary Moments to Wind Moments.

[57] *Bulletin 80*, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

of the same sign as the primary moments. If the first secondaries are 50% of the primary moments then the second secondaries will be about 25% of the primary moments, which will make the resulting moments 75% of the original moments instead of 50 per cent. In general, the first secondary moments will give results which, for girders, are on the unsafe side.

The writers made calculations on two bents: One, the twenty-story Wilson-Maney bent, gave negligible secondary moments; the other, which was obtained by adding 22-ft. bays to the upper ten stories of the Wilson-Maney bent, gave interesting results. Under wind loads the columns of this bent were alternately in tension and compression. Fig. 24 gives the ratio of the first secondary to the original wind moments (lower numeral), and the ratio of the secondary moment after two corrections (algebraic sum of secondary plus first correction plus second correction) to the original moment.

H. V. Spurr,[58] M. Am. Soc. C. E. (by letter).[58a]—The second progress report of the Sub-Committee is in excellent form, and several of the recommendations therein are important changes from the first progress report published in 1931.[59] The recommended wind load remains unchanged, but both the strength factor and the stability factor have been increased, the former by a reduction in the recommended unit stress under wind alone, and the latter by an increased margin of dead load in the columns over and above the uplift due to wind if anchorage is omitted. These are steps in the right direction which will tend to increase rigidity, especially in high narrow buildings, in which the beam sizes will be governed largely by the wind analysis. With lower unit stresses under wind alone the beams will be stiffer, and there will be a general tendency toward deeper connections. Both the strength factor and the stability factor are conservative.

It is still the writer's opinion (as it was in 1931), that the recommended wind load produces unnecessarily heavy shears in the lower stories. Therefore, he recommends a modification of the wind load as follows: For buildings of any height the wind-load diagram may be reduced in the lower third of the height. Starting at the level of one-third of the height of the building, the wind load from that point may be reduced uniformly to zero at the ground level.

This triangular loading on the lower third of the building will result in considerable saving in the web system. The writer believes it is important to consider this modification, because the final recommendations of this Sub-Committee will be given considerable weight by law-making bodies and other technical committees. There is a tendency to exaggerate the effect of wind in the lower parts of buildings, and the simultaneous application of maximum wind load on the full expanse of a building is extremely improbable, if not impossible, because of the turbulence of natural air currents, especially at or near the surface of the earth. A reasonable wind load is important as a basis for sound analysis and for economical design.

[58] Chf. Engr., Purdy & Henderson Co., New York, N. Y.

[58a] Received by the Secretary May 13, 1932.

[59] *Civil Engineering*, March, 1931, p. 478.

An important part of this report outlines the Sub-Committee's recommendations for the analysis of building frames for stresses due to wind when shallow connections are used throughout. The method outlined is a modification of that first proposed[60] by Hardy Cross M. Am. Soc. C. E., for the analysis of continuous frames, in which all joints are fixed in position against displacement. The Cross method has been modified, but not changed in its essential character, by certain semi-graphical steps to reduce the number of cycles necessary for solution. It is proposed for the analysis of existing frames and not for the design of the frame.

It is evident from the text of the report that the Sub-Committee has not covered the ground as to the applicability of the Cross method to the analysis of building frames of all heights and proportions. Some uncertainty is quite natural since the method has a limited usefulness in wind analysis, which is apparent from the fundamental relations in mechanics on which the method itself is based. It can not make an indeterminate structure determinate, nor satisfy the conditions of minimum work which must be fulfilled in any true analysis of a frame in which the joints are free to move in any direction. There should be no confusion on this point, and to assume that the movement of the joints due to direct column stresses will have little or no importance may lead to astounding errors.

On the other hand, if it is realized that the displacement of the joints may profoundly affect the analysis of a badly proportioned frame, and an attempt is made to apply successive corrections, one may become almost hopelessly involved. The adjustment of a continuous frame of many stories under lateral forces produces an elastic balance between vertical and horizontal members under bending and direct stress which is extremely complicated unless the frame is carefully proportioned. Therefore, the writer believes that the Cross method is not suitable for such a problem of analysis. This conclusion has not been reached in any casual manner, but only after the design and analysis of many bents of different proportions ranging in height from 400 to 1 400 feet.

Such studies produce much useful data, including the part played by the columns and beams in the distortion of the web system, the action of columns under axial (vertical) wind stress, and their mutual relations. It becomes evident that as the height of the building increases the effect of column flexure becomes less important as a factor in the flexibility of the web system, and that the changes in length of the columns under axial wind stress become of prime importance. These are outstanding facts

Therefore, it becomes more and more essential, as the height increases, to control the proportions of the web system in order to "maintain a plane after bending," thereby maintaining the position of the joints at any floor in a straight line. This requires a definite distribution of the wind shear in the various panels of a bent, which determines the moments in all the members and controls the design of the beams, whether or not deep connections are used.

[60] *Proceedings*, Am. Soc. C. E., May, 1930, Papers and Discussions, p. 919.

The Cross method is, therefore, somewhat superfluous for design purposes and unnecessarily tedious in the wind analysis of high building frames. The successful design procedure will be based on simple fundamental principles of mechanics without any unnecessary assumptions.

If the web system is properly designed for equal drift in all panels, based on a shear distribution consistent with maintaining a plane after bending, the design analysis will conform with the conditions of least work, and there can be no material deviation, as the structure adjusts itself under load, from the stress conditions of the design analysis. Involved mathematical expressions are not required to demonstrate this truth, which is self-evident from the fact that the structure would be more flexible if any but the design stress conditions should exist.

The writer offers the foregoing comments because the recommended procedure in moment calculations for shallow bracing systems has been based on studies of a 20-story bent which is ideally proportioned. By this is meant that the proportions of the members are generally such as to maintain a plane after bending. This point should be emphasized because the bent in question has been used not only in studies of the applicability of the Cross method, but also in the past in the development of the slope-deflection method. When the various members of a bent are proportioned to maintain a plane after bending, the joints in any floor level will remain in a straight line as the frame distorts under lateral load. In this event the assumption often made, that the change in length of columns under axial wind stress is negligible so far as analysis of the web system is concerned, is correct. It should not be forgotten, however, that this is a special case.

In reviewing any studies made of the Wilson and Maney bent the engineer should keep in mind also that this bent is only of 20 stories, and that the relative proportions of the columns to the beams are quite different from what would obtain in buildings of much greater height. As buildings increase in height the relative stiffness of the columns to beams increases greatly. This change in the proportions of the members has a profound influence on the treatment of the analysis, in that the columns themselves play a more and more minor part in the distortion of the web system as revealed by the slope-deflection method or by the Cross method.

The moments of inertia of the columns increase more rapidly than the column areas increase as the analysis proceeds downward from the top of a high building. In other words, the stiffness of the columns increases more rapidly than the wind shears. This effect is further augmented by the fact that, as the shears increase, with the analysis proceeding downward, the beams generally increase in depth, and the connections tend to become deeper than the beams. This accentuates the increase in column stiffness, because the ratio of the clear span of the columns to story height decreases much more rapidly than the ratio of the clear span of the beams to the span center to center of columns. In a high building, therefore, it is essential, even where shallow bracing is used, to arrive at the stiffness factor of both beams and columns on the basis of their clear spans. If this is not done an exaggerated

importance will be given in the analysis to the elastic influence of the columns in flexure.

A careful analysis of these facts, based on actual studies of high building frames—in which the columns are proportioned for live load and dead load requirements, and the beams are proportioned for the same requirements, or for the requirements of the wind analysis based on the specification for strength as recommended by the Sub-Committee in its report—will disclose, where shallow bracing is used, that the distribution of the shears to the various columns will be governed by the relative stiffness of the beams when the analysis is based on the joints in any floor remaining in a straight line. This means that the points of inflection in the beams are practically at mid-span, even if the ratios of shear in the columns vary materially from the ratios of the column stiffness values. This is easily visualized by the fact that the columns are so stiff that they contribute little to web deflection, and any variation in the small amount that they do contribute, has little effect. If the columns were infinitely rigid this truth would be self-evident. When the beams are relatively several times more flexible than the columns they become (through slight movements of the points of inflection from mid-span) very sensitive instruments of adjustment.

The writer has made studies of the application of the Cross method, as recommended by the Sub-Committee, to an 80-story bent with beams of markedly different spans. Fig. 25(*a*) is a diagrammatic sketch of a typical bent at the twentieth floor. The characteristics of its members are given in Table 3. The diagram illustrates the application of the Cross method to live load and dead load moments in a rigid bent.

The solution for moments due to live and dead loads on the beams, indicates very clearly the great simplicity and usefulness of the Cross method when applied to this type of load, for which there is no movement of the joints.

TABLE 3.—Make-Up of Columns and Beams in Fig. 25

Member	Section	Stiffness factor, $\frac{I}{L}$	Member	Section	Stiffness factor, $\frac{I}{L}$
Column 1	14-in., 427-lb. H-beam two 24-in. by 3½-in. plates...	12.4	Column 3	14-in., 427-lb. H-beam two 24 by 4-in. plates.........	15.6
			Beam 1–2	33-in., 125-lb. Bethlehem...	1.57
Column 2	14-in., 427-lb. H-beam two 24-in. by 3½-in. plates...	12.4	Beam 2–3	22-in., 67-lb. Bethlehem...	1.00
			Beam 3–4	36-in., 260-lb. Bethlehem girder..................	3.54

The panel shears are those that will maintain a plane after bending. For beams, L is the full span from center to center of columns and for columns, it is the full story height. The values of $\frac{I}{L}$ are relative (see Table 3). The beams are designed so that $\frac{I}{L}$ is approximately proportional to the panel

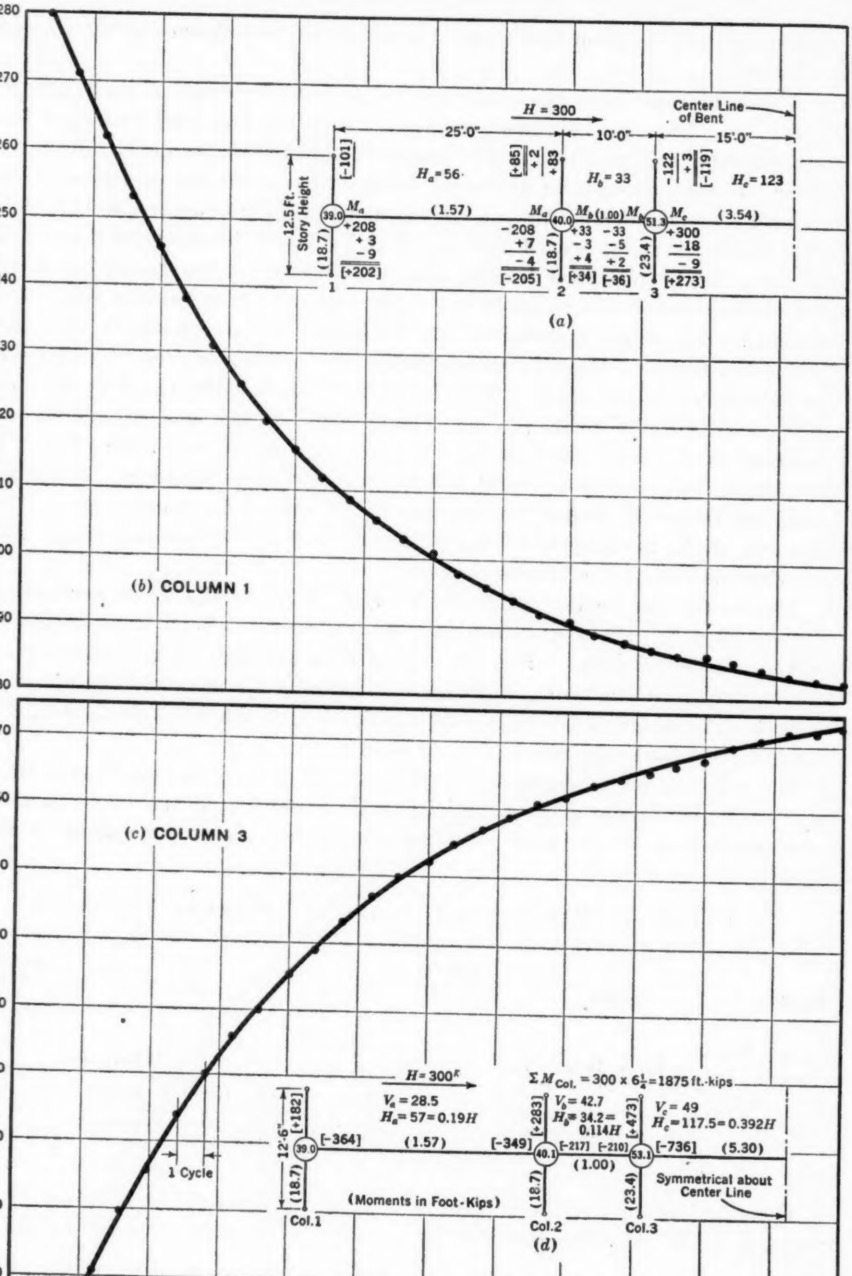


FIG. 25.—CROSS METHOD APPLIED TO WIND-BRACING ANALYSIS.

shear. The fixed-end beam moments, M_a, M_b, and M_c ($= \frac{1}{12} wL^2$), are for a uniform load of 4 kips per ft. The relative $\frac{I}{L}$ values are shown (Fig. 25) in parenthesis and the final moments are in brackets.

In all the analyses the symmetry of the bent and the similarity of conditions from story to story are utilized in order to simplify the calculations. One story of the bent is isolated, points of inflection in the columns being taken at the mid-point of the story. For wind analysis a point of inflection is taken at the middle of the center beam span. There are, of course, no carry-over moments to these points, and the stiffness factors for the members are three-fourths of what they would be for fixed-end members of the same length.

A complete analysis of the bent, for wind loads, was made for purposes of study and comparison. The solution was carried as many as 30 cycles to a point at which the change in column moments with each cycle was less than 0.5 per cent. In Fig. 25(*d*) the final moments are shown in brackets, but the various steps have been omitted for brevity. The moments in Columns 1 and 3 after each successive cycle are plotted in Figs. 25(*b*) and 25(*c*), the ordinates representing the moments and the abscissas the number of cycles completed. There is practically no change in moment in Column 2 throughout the cycles of solution.

Because of the process followed in the proposed adaptation of the Cross method, it is evident that the number of cycles required for a solution increases with the stiffness of the columns relative to the beams; that is, the number of cycles required depends upon the percentage of the column moment subtracted when the joints are released and the moments distributed.

It is apparent from a casual inspection of Figs. 25(*b*) and 25(*c*) that the use of the Cross method in the form recommended by the Sub-Committee will be very laborious in the analysis of high building frames, because of the great stiffness of the columns relative to the stiffness of the beams. This relation necessitates a large number of cycles, which will prove very tedious to carry out to the extent which accuracy will demand. The Sub-Committee expected that the semi-graphical method proposed would shorten the time and labor required, but its use will generally introduce larger errors than would obtain by not using the method at all. This is due to the fact that the columns play such a minor part in the distortion of the web system that the location of the points of zero moment is only slightly affected by column bending, and this small influence can not be traced by rough approximations. Accuracy under these circumstances would generally be better served by ignoring column bending entirely by distributing at once the total floor moment between the beams in proportion to their relative stiffness.

The process of distributing end moments as used in the case of Fig. 25, is in reality one of progressive distortions, which is rather difficult to visualize unless the points of inflection in the columns are considered as pinned down in space before the joints are released in each cycle of adjustment. This con-

ception, or its equivalent, is also necessary to maintain the structure in equilibrium during the process of end-moment distribution.

The writer, therefore, proposes a simplification of the Cross method, which he believes is particularly useful in the analysis of high building frames when typical tier construction is used. The method is illustrated in Fig. 26 for the same example as is worked out in Fig. 25(*d*), and the saving in time and labor is apparent. The same column sections and column moments are used above and below the floor. As in the previous case, $H = 300$ kips; then, $\frac{1}{2} \Sigma M_{col} = \frac{1}{2} (300 \times 6\frac{1}{4}) = 937.5$ kips; and, $f = \dfrac{937.5}{313.2} = 2.99$, in which f is a constant.

In the first place, in typical tier construction in high buildings the floor-beams gradually increase in strength from top to bottom to meet the requirements of wind moments. There are no sudden jumps in sizes and proportions

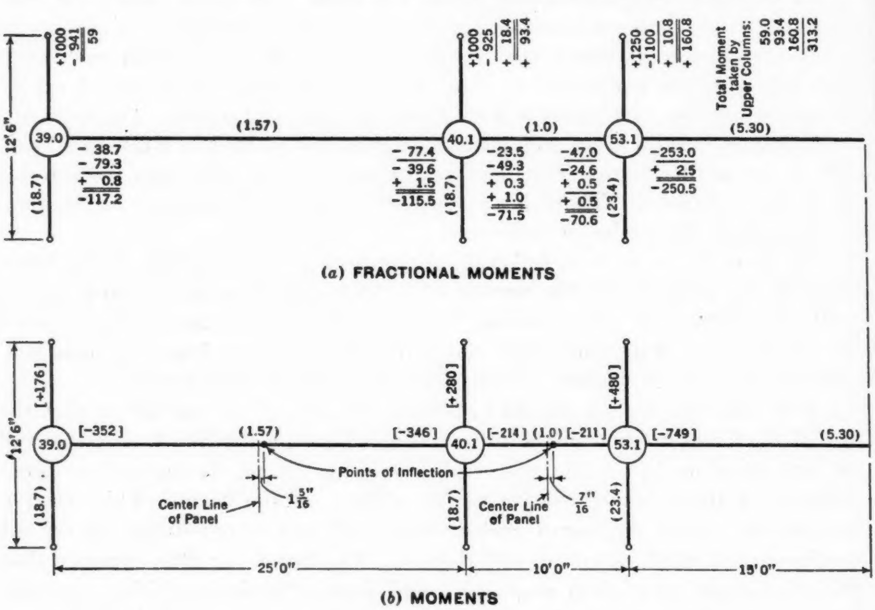


FIG. 26.—WIND-ANALYSIS BY A SIMPLIFIED CROSS METHOD.

from one floor to the next. Furthermore, the columns are gradually gaining in stiffness relative to the beams, because of live load and dead load requirements. It is not difficult to visualize or to demonstrate that under these circumstances the points of zero moment in the columns are practically at the mid-points of their clear spans, and that the elastic adjustment is accomplished in the beams by the movement of their points of zero moment away from the stiffer and toward the more flexible of the two columns which the beams, respectively, join together.

Thus, the first simplification is obtained, and one may accept the diagrams shown in Fig. 25(*d*) and Fig. 26, in which the points of inflection in the

columns are shown at the middle of their clear spans. The columns and beams form an elastic system which will respond to adjustment under a partial load in the same manner as it would under a full load. Next, this system may be tested by analysis, thus avoiding the trouble of making the model and test observations, as follows:

Step (1).—Apply 1 000 ft-kips to the most flexible column above and below the floor line, and distribute moments to all the other columns in the ratio of their K values. This may be visualized as if it were accomplished by horizontal forces acting at the top and bottom of the columns at their points of inflection, while all the joints at the floor line are considered fixed against rotation. These horizontal forces acting from left to right at the top of the columns, and from right to left at the bottom of the columns, will deflect all the columns an equal amount and produce unbalanced moments at all joints.

Step (2).—The ends of the columns are now considered pinned in position against translation.

Step (3).—The joints are next released and the end moments distributed by the Cross method. Usually three cycles will give ample accuracy.

Step (4).—The moments in all the columns are then added together and their sum compared with the total floor moment required for wind. In other words, find the ratio of the total wind moment at the floor to the sum of the column moments after Step (3).

Step (5).—This comparison (Step (4)) will give a constant by which all the trial moments are multiplied to obtain the true moments.

In the example given in Fig. 26 the work has been simplified by using in each column applied moments of the same amount above and below the floor line.

In an actual building the wind shear would be larger below the floor than above it, as a general condition. The method in its simplified form may still be used under these circumstances, however, as follows:

Steps (1), (2), (3), and *(4).*—These steps would be unchanged.

Step (5).—Multiply all the beam moments after Step (3) by the ratio obtained from Step (4), to obtain the true beam moments.

Step (6).—Find the ratio of the total wind moment above the floor line to the sum of the column moments after Step (3) above the floor line.

Step (7).—Multiply the column moments above the floor line after Step (3) by the ratio obtained from Step (6), to obtain the true column moments above the floor line.

Step (8).—Find the ratio of the total wind moment below the floor line to the sum of the column moments after Step (3) below the floor line.

Step (9).—Multiply all column moments below the floor line after Step (3) by the ratio obtained from Step (8), to obtain the true column moments below the floor line.

In an actual analysis of an existing structure this simplified method could be applied at any intermediate floor, and the effect of column bending on the

location of the points of inflection in the beams determined. If it were found that the points of inflection in the beams remained practically at mid-span, it would mean that this operation need not be repeated in the typical floors below, unless there were some abnormal change in the proportions of the structure. This would eliminate a tremendous amount of work, as the total floor moments could be distributed between the beams in proportion to their rigidities.

The reason why it would not be necessary to investigate the typical floors below the trial level is that, because of the increasing relative stiffness of the columns as the analysis proceeds downward, the movement of the points of inflection in the beams from mid-span position would be less than in the floor investigated. By successive trials at higher levels the general location of the points of inflection in the beams will be determined throughout the structure, and one solution will easily serve for several adjacent floors. In this manner a rapid analysis may be made of the bending moments in the beams and columns, based on the joints at any one floor level remaining in a straight line. Throughout these operations the writer believes that accuracy would be increased by calculating the K values of all members on the basis of their clear spans.

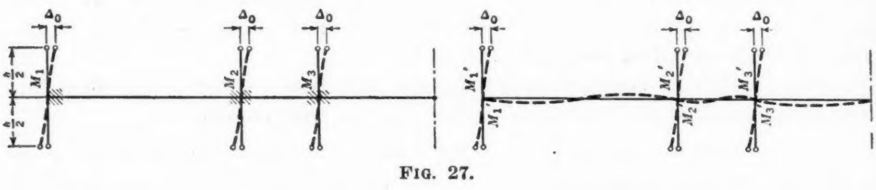


FIG. 27.

Reference is made to Fig. 26 and the explanation that followed. In Fig. 27(a), the applied moments M_3 M_2, and M_1, are such that $M_1 : M_2 : M_3 = K_1 : K_2 : K_3$, in which, K_1, K_2, and K_3 are the $\frac{I}{L}$ values for the columns. The diagram shows the initial distortion of the columns under any fractional loads that produce a common deflection, Δ_0, with the joints temporarily fixed against rotation.

The ends of the columns are next pinned in position. The joints are then released and the moments distributed, which results in the condition shown in Fig. 27(b). Actually, the yielding of the beams has reduced the moments originally induced in the columns when the joints were held against rotation. At the same time the value of Δ_0 is unchanged by condition.

The elastic system is now adjusted to the definite distortion, Δ_0, and M_1 has changed to M'_1. One may easily calculate the value of Δ_0, the original distortion in the columns, from the value of M_1. The wind load moments may be obtained, as previously explained, by proportion.

The wind moment in the left-hand column may be designated as M_w, and the drift in the web system under the assumed wind load can be found by simple proportion as follows: Let Δ_{web} equal the total drift from the

distortion of the web system. Then $\Delta_{web} = \dfrac{M_w}{M'_1}\Delta_0$. This solution is easily visualized and can be obtained quickly by slide-rule. It can be applied to any combination of spans, but depends for its correctness upon the conditions, that the joints remain in a straight line, and that the points of inflection in the columns are correctly chosen.

This leads to Fig. 28(*a*), which is drawn to indicate the influence of the difference in shear above and below the floor line on the location of the point

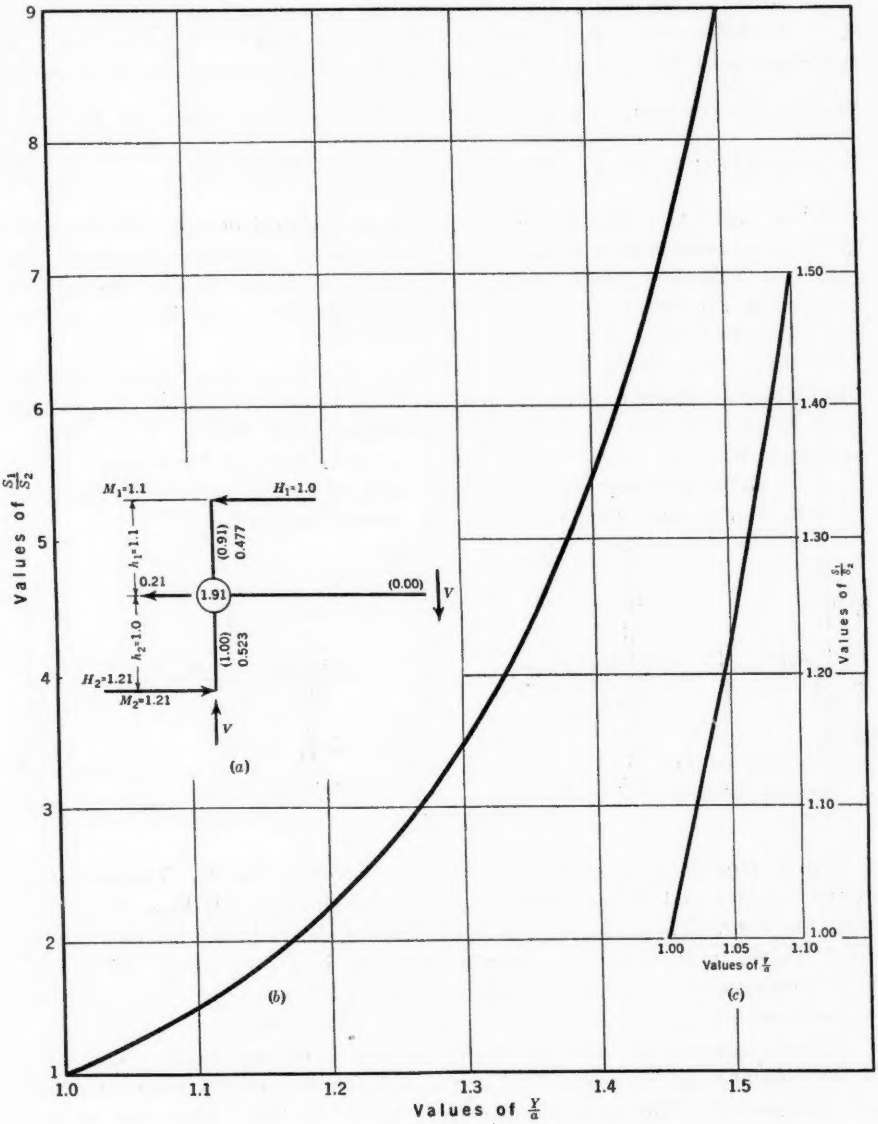


FIG. 28.

of inflection in the column. Given a shear of unity above the floor, and below the floor a shear of 1.21, Fig. 28(*a*) indicates that the distances to the points of inflection would be inversely proportioned to the square root of the shears. This relation of arms and reactions may be proved by the Cross method, but in an actual frame it would be upset if the beams in adjacent floors were not designed to maintain a constant drift.

This relation is plotted in Fig. 28(*b*), in which, a represents one-half the clear story height of the column, and y, the value, a, plus the amount the point of inflection would move up from the mid-story point. It is evident that the difference in shear above and below the floor has a minor influence a few stories below the top. Ten stories down the difference in shear would be about 10 per cent. Then, $\frac{y}{a} = 1.025$. Twenty stories down the difference in shear would be 5% and $\frac{y}{a}$ would equal 1.013. These values are so nearly equal to unity that they may be neglected in practical design, and attention given to maintaining a uniform drift from beam distortion, based on the points of inflection in the columns being at mid-story. Hence, the method outlined in Fig. 26 may be followed (provided the joints remain in a straight line) for the typical floors of a high tower.

In Figs. 25 to 28 the analysis of the web system has been treated without regard to chord action. No attempt has been made to visualize the vertical loads induced in the columns by the horizontal wind forces. It is evident, of course, that the overturning moment must be resisted by vertical forces in the columns, and that these forces are induced by mutual reaction between columns and beams. One can not exist without the other.

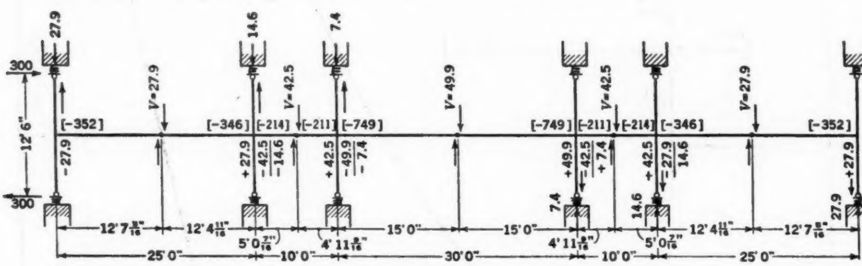


FIG. 29.

Fig. 29 represents the one story of the bent solved in Fig. 26. The moments in the columns and beams are based on a wind shear of 300 kips above and below the floor. These moments produce definite shears in the beams as indicated, which, in turn, produce definite reactions in the columns. The reactions form couples which balance the moment of 3 750 ft-kips. The resisting moment couples are: 7.4×30; 14.6×50; and 27.9×100.

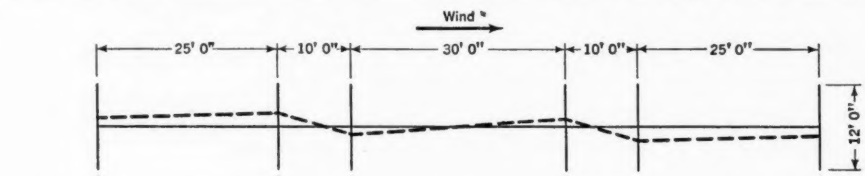
The six columns and five beams may be considered as a model. The ends of the columns are on rollers and are held between supports vertically, but not horizontally. The supports are held exactly in line. The ends of the

columns are considered as pin-connected to a link of unchanging length under stress. Forces of 300 kips applied from left to right at the top of the columns, and 300 kips from right to left at the bottom of the columns, produce the conditions of stress indicated in beams and columns and the vertical reactions noted at the ends of the columns. All the columns, beams, and supports have played their part in this result, and no member can be changed without modifying the stress conditions in all the members. Similarly, no support can be removed or moved, that re-acts against the end of any column without a definite change in the moments in all the members. When the moments are modified the reactions at the column ends are modified, and *vice versa.*

The change in length of the columns under direct stress for one-half the story height may be neglected in this example because the column areas are such as to maintain the joints in a straight line, but the floor is slightly tilted by the columns changing length under chord action.

If fifty floors were superimposed like fifty models, as shown in Fig. 29, then, although any floor might be analyzed separately as in Fig. 26, the solution would not be correct unless each floor was proportioned properly. This follows quite naturally, since every floor is inducing vertical loads into the columns and affecting the supports of every other floor.

If various floors are proportioned in a haphazard manner and analyzed separately, one may calculate the vertical loads in the columns between any floor and rigid foundations, and estimate the relative position of the joints in the floor in question. Such a condition is illustrated in Fig. 30, in which,



Panel H's	1.00		1.00	1.00		1.00	1.00	
Panel V's	1.00		2.50	0.83		2.50	1.00	
Column V's	+1.00	+1.5		−1.67	+1.67		−1.50	−1.00
Column Areas	1.0	1.0		1.12	1.12		1.0	1.0
Column $\frac{V}{A}$	+1.00	+1.50		−1.50	+1.50		−1.50	−1.00

① ② ③ ③ ② ①

FIG. 30.—TYPICAL FLOOR, SHOWING RELATIVE MOVEMENT OF JOINTS DUE TO AXIAL COLUMN DEFORMATION.

the shear is assumed to be equally distributed in the five panels, and the beams are proportioned for this condition by the method given in Fig. 26. The bents are 20 ft. apart and subjected to a wind load of 20 lb. per sq. ft. For 80 stories of height at 12.5 ft., therefore, the moment due to wind is 200 000 ft-kips. Furthermore, $100\ V_1 + 1.5\ V_1 \times 50 - 1.67\ V_1 \times 30 = 200\,000$, and, $V_1 = 1\,600$ kips on 380 sq. in. = 4.2 kips per sq. in. Similarly, V_2 and V_3 equal 6.3 kips per sq. in. Chord deflections, Δ, are computed as follows:

Bent 1-1:

$$\Delta_{1\text{-}1} = 0.001 \times \frac{4.2}{4.5} \times \frac{10}{10} \times 1\,000 = 0.93 \text{ ft. with the wind;}$$

Bent 2-2:

$$\Delta_{2\text{-}2} = 0.001 \times \frac{6.3}{4.5} \times \frac{20}{10} \times 1\,000 = 2.80 \text{ ft. with the wind; and,}$$

Bent 3-3:

$$\Delta_{3\text{-}3} = 0.001 \times \frac{6.3}{4.5} \times \frac{33}{10} \times 1\,000 = 4.65 \text{ ft. against the wind.}$$

The foregoing values of chord deflections are a measure of inconsistency in the method. The building would have to fly to pieces to produce these results.

Similarly, it is evident that the movement of the joints as indicated would reduce the shear resistance in the 10-ft. panels and increase it in the others as compared with the assumed distribution of shear. In other words, the structure will adjust itself under load to reduce the amount of this action. This adjustment may be very complex in a high tower, but is always in the direction of maintaining a plane after bending. An exact analysis under these conditions is scarcely possible.

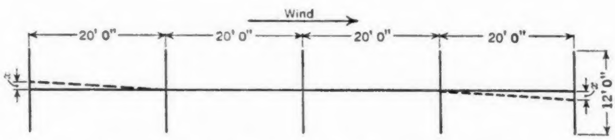


FIG. 31.—TYPICAL FLOOR SHOWING VERTICAL DISPLACEMENT OF JOINTS DUE TO AXIAL COLUMN DEFORMATION.

The usual portal method of shear distribution is shown in Fig. 31. The shears are assumed as equal in each panel and the beams are proportioned accordingly, for a 40-story building, 480 ft. high:

At the bottom,

$$M_w = (20 \times 20 \times 480)\,240 = 46\,100\,000 \text{ ft-lb.;}$$

$$V = \frac{M_w}{80} = 580\,000 \text{ lb.;}$$

Column area = 145 sq. in.; and,

$$f = \frac{V}{A} = 4\,000 \text{ lb. per sq. in.}$$

At the forty-first floor,

$$x = \frac{1}{2} \times 480 \times 12 \times \frac{4\,000}{E} = 0.40 \text{ in.}$$

At the thirty-first floor,

$$x = \frac{5}{4} \times \frac{3}{4} \times 0.40 \text{ in.} = 0.375 \text{ in.}$$

At the twenty-first floor,

$$x = \frac{3}{2} \times \frac{1}{2} \times 0.40 = 0.30 \text{ in.}$$

At the eleventh floor,

$$x = \frac{7}{4} \times \frac{1}{4} \times 0.40 = 0.175 \text{ in.}$$

It is to be noted that, for a web deflection equal to $0.001\ h$, $x = 0.24$ in. gives 100% loss of rigidity in the exterior panels. (See Fig. 32.)

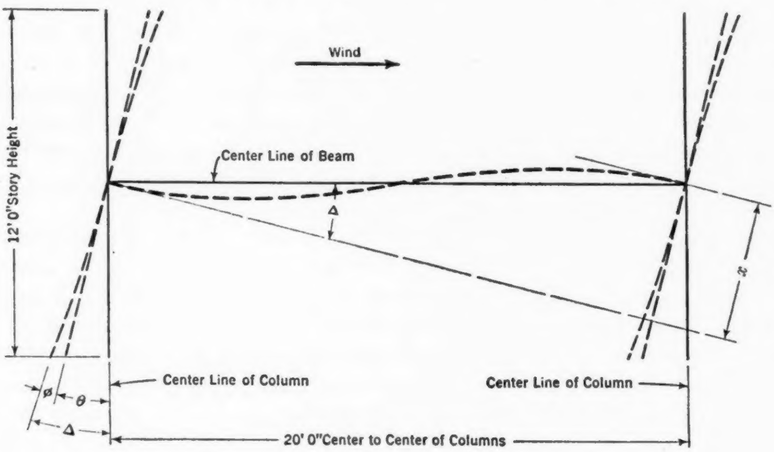


FIG. 32.

Chord action will greatly modify the stress conditions in a high building. The rigidity of the outside panels will be reduced by chord action, and the inside panels will have more than 25% of the total shear. The exact amount will vary from floor to floor. Furthermore, the structure will adjust itself "wisely" and the inside columns will have some direct stress.

Inconsistencies for a 40-story office building of normal construction are listed in Table 4. The story height is 12 ft. and the bents are spaced at 20 ft. Half-way up the building the chord action would be such under the

TABLE 4.—INCONSISTENCIES FOR A 40-STORY OFFICE BUILDING OF NORMAL CONSTRUCTION. WIND AT 20 POUNDS PER SQUARE FOOT

Number of floor	x, in inches	Percentage, inconsistency in exterior panel
41	0.400	167
31	0.375	156
21	0.300	125
11	0.175	73
1	.....	...

shear distribution assumed, that the outside panels would be slacked off 125%; in other words, they would be helping the wind, and not resisting it. This would result in the middle panels taking 2.25 times the amount of shear assumed. Such a condition of stress is impossible. The building adjusts itself in a complex manner toward maintaining the joints more or less in line, and the inside panels pick up additional shear.

The mutual relation between chord and web action is shown by Fig. 32. It is self-explanatory. In Fig. 32 the functions indicated are defined as follows: θ = beam deflection; ϕ = column deflection; $\Delta = \theta + \phi$ = web deflection; and, x = equivalent displacement of one end of the beam to produce distortion in the web system equal to the distortion from wind shear. Let $\Delta = 0.001\,h$; $\theta = 0.0009\,h$; and $\phi = 0.0001\,h$. Then, x = span $\times$ tan $\Delta = 20 \times 12 \times 0.001 = 0.24$, or $\frac{1}{4}$ in. The values of θ and ϕ are representative of those that would apply to lower stories of a high building, designed to resist wind under the recommendations of the Sub-Committee. For the case noted the movement of one end of the beam $\frac{1}{4}$ in. affects the shear resistance of the panel 100 per cent. This effect can not be considered as one of secondary stress. It produces an equivalent stress condition, and it must be considered as a primary action that takes place throughout the structure, tending always to reduce the total deflection of the frame.

Figs. 30 and 31 should be reviewed after a study of Fig. 32 has revealed the mutual interacting relation of chord and web members. The analysis of the web system by the Cross method, as shown in Fig. 26, is, of course, based on the analysis of the elastic curve of members under bending. This ultimately concerns itself with the change in length of longitudinal fibers under direct stress. Consistency requires an equal regard for the change in length of the longitudinal fibers in the chord system. An unequal change in length between adjacent longitudinal fibers produces curvature, and a bent can not hold together with a different curvature between pairs of columns in the same bent. Therefore, the columns must slide by one another to correct such a condition of strain, and this action modifies the stress conditions throughout the structure in such a manner as to reduce the amount of this relative slip between the members of a multiple chord system. The balance comes when the condition of minimum deflection for the structure as a whole has been satisfied. Otherwise, a frame structure would be in an unstable condition under internal stress from lateral loads.

Although considerable information may be obtained by the steps outlined in Figs. 26, 27, 28, and 29, the real analysis of a high building frame must still be considered in its preliminary trial stages until the vertical loads induced in the columns by the assumed web analysis have been ascertained. Various "short-cuts" may then be used, at the discretion of the engineer, to determine approximately the effect of the change in length of the columns upon the relative positions of the joints in any floor throughout the structure. If it develops, as a general condition, that the joints would move out of line under the vertical forces induced in the columns by a web analysis which assumed all the joints to be in a straight line, it means that the entire web analysis is inconsistent with the elastic proportions of the structure. It may develop

that this inconsistency is slight, but it is quite probable that it would be large if the designing engineer had proportioned the frame without due regard to the relative stiffness of the various bents between themselves, and the relative stiffness of the various panels in any one bent under chord deflections. The problem under certain circumstances will assume terrific proportions, and for practical reasons it may become imperative for the engineer to use approximations that do not consume an unreasonable length of time. It will require great insight and ability on the part of the engineer to arrive at a reasonable solution in many cases. The general phases of the problem will involve:

1.—The application of the proper wind load to the structure as a whole.

2.—The division of total load between the various bents in proportion to their relative rigidities, approximately estimated.

3.—A preliminary survey of the various bents, as to their probable ability to act together under the division of load assumed.

4.—Determination of the purposes of investigation and analysis: Whether such investigation is for scientific purposes, for the owners, or for other interested parties.

5.—A preliminary survey of the various bents that will necessitate the exercise of considerable judgment regarding the influence of masonry in augmenting the resistance of certain bents.

6.—The question of the general and the individual stability of the various bents.

After all these and similar phases of the problem are considered the analysis may proceed, and the character and accuracy of the methods used should depend upon the importance of the problem.

Conditions that affect the position of the points of zero moment in the members may be reviewed briefly as a guide in analysis:

1.—When there are many stories of equal height, as in the shaft of a high building, the position of the points of inflection in the columns will be affected if there is a sudden jump in the relation of stiffness between the floors. This does not usually happen, and this influence can be prevented in design by making the various floors of equal stiffness under their respective loads. The usual factor that would affect the position of the points of inflection in the columns under these conditions would be the growth in wind shear downward from floor to floor.

The general effect of this inequality in shears is to raise the points of zero moment in the columns slightly above the center of the column spans. It thus produces a very slight increase in the relative flexibility of the columns to the beams from that which would obtain by assuming the points of zero moment in the columns at the center of the span, and tends to reduce the difference between the moments in the columns below and above the floor. Generally speaking, this effect is so minor that it may be neglected, except in the extreme upper stories of a building.

2.—When the columns are relatively much stiffer than the beams, the points of inflection in the beams are practically at mid-span.

If the engineer is concerned with the effect of variation in the rigidity of the beams in adjacent floors, and particularly with the effect of this variation on the points of inflection in the columns; or if he is concerned with the effect of differences in adjacent story heights, whether the floor flexibility is the same or different, the problem is simplified in the lower stories of a high building by the fact the shear may be assumed constant through several adjacent stories in considering the position of the points of counterflexure.

In order to visualize the problem and arrive at a key for determining the elastic conditions involved, turn, for the sake of simplicity, to the study of an exterior column. It may be considered as of uniform section through the stories under consideration, and the cantilevers at the floor lines of equal length, loaded at their ends. This column, for the purposes of study, has a constant shear which would produce an overturning moment at the bottom equal to the shear times its length, except for the interposition at the floor lines of reverse couples, which are applied by the loads at the ends of cantilevers of equal length. The simile is not complete without considering the ends of these cantilevers held a constant distance apart in the actual structure. This is true because the points of inflection when at the center of the beams do not move vertically, except for the movement induced by the tilting of the floors. The slight difference in slope between adjacent floors may be neglected in the study.

It is apparent that if any of these cantilevers were more flexible than others, they could not have an equal restraining action on the column without unequal amounts of internal work. With equal moments in the cantilevers the more flexible ones would deflect the most, and the sum total of the internal work in all cases would be larger than would be the case if each cantilever picked up restraining moments proportionate to their rigidities. With the cantilevers acting in the latter manner they would produce the greatest restraint in the column with the total amount of internal work a minimum within themselves.

The minimum of internal work in the column will be performed when the points of zero moment in the column are at the mid-story points. Therefore, if the cantilevers are proportioned so that their rigidities vary in the same ratio as the magnitude of the moments which would be induced in them with the points of inflection in the column at the mid-story points, then the points of zero moment will be at the mid-story points, because the condition of minimum work in the elastic system has been fulfilled.

A condition similar to that described in the foregoing comments is demonstrated by Fig. 33, in which a single column of constant section is framed to eight cantilevers of equal length, representing one-half the floor-beams, with their points of counterflexure at mid-span. The story heights vary. The stiffness factors of the various members are shown in parentheses. The rigidities of the cantilevers are proportional to the magnitude of the moments produced by a constant shear in the column with the points of inflection at mid-story.

This is a definite elastic system, that can be tested as if it were a model. Assume the upper and lower ends of the column pinned in position against

translation. Deflect the free end of all the cantilevers upward an equal amount, with all joints of cantilevers to column held fixed against rotation. The moment induced in the cantilevers will be proportional to their rigidities, and these moments are given in brackets. They are noted on Fig. 33 at the free end of the cantilevers for convenience, but, of course, they represent the moments induced in the cantilevers at the fixed joints. The free ends of the cantilevers are now pinned in position. The joints are next made free to rotate and the moments distributed by the Cross method, considering that all joints remain in a straight line. The resulting moments in the beams are indicated at the left-hand end of the beams, and the various moments in the columns are indicated above and below the floors, along the column. In the various stories the sum of the column moments, as well as the total shear, in that story are indicated. It will be noted that the shear is not constant. This demonstrates that in an actual structure so proportioned a constant horizontal shear could not be resisted and still maintain the joints in a straight line.

An actual structure, proportioned as indicated, under distortion from a constant wind shear considered as a force applied at the top of the column from left to right, would be free to move horizontally in order to take up the proper shape to resist the shear. Conditions of stress for such a loading can be revealed by a further application of the Cross method, the procedure being to apply additional moments to the columns in the stories where the shears are low, as indicated in Fig. 33, by successive cycles until the shear is of the same value in all stories. If this is done it will be found, for this particular structure, that the points of inflection in the columns are at their mid-span points, and that the moments in the cantilevers are proportional to their rigidities. This demonstrates how to proportion the beams in the lower stories of a high building to meet the condition of unequal story heights, and have the moments of design conform with the true moments.

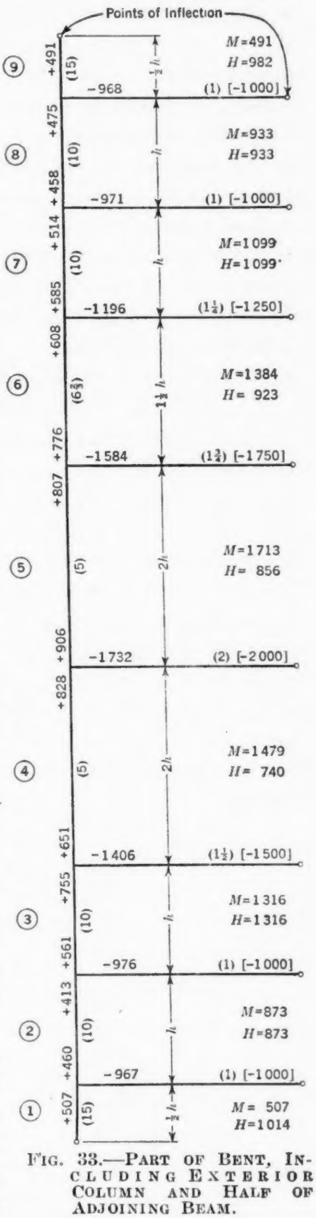


FIG. 33.—PART OF BENT, INCLUDING EXTERIOR COLUMN AND HALF OF ADJOINING BEAM.

It should be evident, from what has gone before, that from the standpoint of design the golden moment for the engineer is when he is proportioning the

structure for analysis, if there is to be any practical attempt on his part to have the calculations represent the true conditions of stress. If proper care is taken at the beginning of the design analysis to proportion the members of the web system so as to maintain the joints in the various floors in a straight line, the problem may be broken into rather simple elements. Conversely, it should be apparent that if the structure is badly proportioned, the difficulties of analysis are tremendously increased, and may become insurmountable from a practical standpoint. Furthermore, it should be evident that it is rather useless, from a practical standpoint at least, to develop mathematical "straight-jackets" which depend for their workability upon assumptions which violate the true conditions of elastic behavior. Such mathematical processes blind the operator to true conditions and are misleading. It seems far better to break the problem down into its true elements and meet the difficulties of analysis as honestly as possible. By so doing perception and insight will be trained and judgment developed.

The problem of designing high towers is tied up with column action, not only as regards wind analysis, but as regards the design for the loads of construction and occupancy. As the height of the building increases it becomes more important to consider the question of designing the columns on a dead-load basis to avoid differentials in change of length which may accumulate in very high structures to a serious degree.

Furthermore, such structures, due to the requirements of wind analysis, become continuous under live and dead load. This raises questions in design which ultimately will have to be faced and given careful consideration. They involve a thorough study of various types of connections and their efficiency under all conditions, together with the influence of possible relative movements in the foundation supports. All these are major problems which greatly exceed in importance the question of exact precision in mathematical analysis.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from August 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ABBEY, CHESTER EDWARD, St. Louis, Mo. (Age 21). Refers to E. O. Sweetser, J. L. Van Ornum.

AGONIAS, EUSEBIO MALVAR, Rolla, Mo. (Age 24). Refers to C. E. S. Bardsley, J. B. Butler, E. W. Carlton, E. G. Harris.

AHRENS, HERBERT EMMETT, Rolla, Mo. (Age 35). Asst. Prof., Missouri School of Mines. Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, W. S. Gearhart, E. G. Harris, C. M. Slaymaker.

ALBERTIS, ALEXANDER JOHN, Urbana, Ill. (Age 23). Refers to J. S. Crandell, W. C. Huntington, C. E. Palmer.

ALEXANDER, JAY, Dallas, Tex. (Age 36). Civ. and Mech. Engr. Refers to C. T. Bartlett, F. W. Cawthon, O. N. Floyd, J. D. Fowler, G W. Hamilton, A. W. Hardy, J. B. Hawley, J. A. Norris, J. W. Pritchett.

ALLEN, LLOYD LEE, Philomath, Ore. (Age 25). Refers to J. R. Griffith, H. S. Rogers.

ALLMOND, DAVID ROBINSON, JR., St. Louis, Mo. (Age 33). Sales Representative, American Bitumuls Co. Refers to A. B. Brown, J. F. Davidson, L. D. Draper, P. L. Fahrney, W. M. Francis, H. K. Preston, W. J. Wagner.

ALOFF, ABRAHAM MARTIN, Dorchester, Mass. (Age 24). Jun. Examiner, Div. of Civ. Service, Commonwealth of Massachusetts. Refers to E. S. Dorr, A. Haertlein, L. J. Johnson, L. J. Phillips.

ANDERSEN, ROY GIHM, Portland, Ore. (Age 22). Refers to J. R. Griffith, H. S. Rogers.

ATTRIDGE, WILLIAM JAMES, Rigby, Idaho (Age 28). Refers to I. N. Carter, I. C. Crawford, J. W. Howard.

BACKMAN, JOHN EDWARD, Sea Isle City, N. J. (Age 21). Refers to E. F. Berry, E. F. Church, L. Mitchell, S. D. Sarason.

BAIRD, DOUGLAS GEORGE, Portland, Ore. (Age 22). Refers to J. R. Griffith, H. S. Rogers.

BAKER, JAMES LEROY, Corsicana, Tex. (Age 22). Plant Inspector, Texas State Highway Dept. Refers to M. L. Bowers, R. C. Gans, E. J. McCaustland, H. K. Rubey.

BARCLAY, ROBERT KENNEDY, Oakmont, Pa. (Age 22). Refers to A. Diefendorf, L. C. McCandliss.

BARKER, CARL LEON, Birmingham, Ala. (Age 30). Refers to J. G. Allen, G. J. Davis, Jr., D. C. A. duPlantier, N. E. Lant, D. B. Rush.

BARREKETTE, ABRAHAM ELIEZER, Jerusalem, Palestine (Age 33). Engr. and Contr. Refers to P. H. Budd, H. Grand, I. Gutmann, H. P. Hammond, E. J. Squire.

BARRON, EDGAR GORDON, Trenton, N. J. (Age 25). Asst. Engr., U. S. Geological Survey, Water Resources Branch. Refers to H. T. Critchlow, S. B. Folk, O. W. Hartwell, O. Lauterhahn, L. Lee, A. Richards.

BATES, ABEL JACOB, Webster, Mass. (Age 22). Refers to A. Haertlein, L. J. Johnson.

BAUGH, ELBERT ANSEL, Dallas, Tex. (Age 34). Gen. Mgr., Texas Branch, Associated Gen. Contrs. Refers to F. N. Baldwin, W. P. Bentley, O. H. Koch, T. G. MacCarthy, E. N. Noyes.

BELZ, CHARLES JOHN, Dayton, Ohio (Age 40). Asst. Prof. of Civ. Eng., Univ. of Dayton. Refers to E. O. Brown, J. J. Chamberlain, Jr., W. E. Keyser, R. B. Prinz, B. T. Schad.

BENNETT, NELSON CLARK, Brooklyn, N. Y. (Age 21). Refers to H. R. Codwise, H. P. Hammond, L. F. Rader, E. J. Squire.

BENSCOTER, STANLEY URNER, Kansas City, Mo. (Age 21). Refers to H. Cross, J. J. Doland.

BERKENBOSCH, JOHN CARLON, Kirkwood, Mo. (Age 22). Refers to C. E. S. Bardsley, J. B. Butler, E. W. Carlton, E. G. Harris, R. W. Jablonsky.

BIRKELAND, HALVARD WESSEL, Seattle, Wash. (Age 24). Refers to I. L. Collier, G. E. Hawthorn, S. Ivarsson, C. C. More, R. G. Tyler.

BLAIN, WILBER ALEXANDER, Mercer, Pa. (Age 23). Refers to F. J. Evans, F. M. McCullough, J. H. Robinson, C. B. Stanton, H. A. Thomas.

BLISS, PERCY HENRY, Vancouver, Wash. (Age 24). Refers to A. H. Beyer, D. M. Burmister, W. J. Krefeld.

BODDINGTON, NORMAN, Punjab, India (Age 32). Capt., Royal Engrs.; Subdivisional Officer, Hydro-Elec., Branch, Punjab Public Works Dept. Refers to D. S. McPhail, W. G. Wheatley. (Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

BOGART, DEAN BUTLER, Bloomfield, N. J. (Age 22). Refers to H. N. Cummings, W. S. LaLonde, Jr.

BONNELL, JOHN CALVIN, Morris, Ill. (Age 27). Supt., Congress Constr. Co. Refers to C. R. Andrew, R. A. Bonnell, Sr., L. D. Cornish, J. A. Harman, W. C. Weeks.

BOOKMAN, MAX, Los Angeles, Cal. (Age 21). Refers to C. Derleth, Jr., B. A. Etcheverry, B. Jameyson.

BOOTHE, PERRY MATTISON, Los Angeles, Cal. (Age 22). Refers to F. J. Converse, R. R. Martel, W. W. Michael, F. Thomas.

BOWMAN, EDWARD KNOTT, Olympia, Wash. (Age 22). Refers to I. L Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

BOYLAN, FRANK PATRICK, Philadelphia, Pa. (Age 24). Refers to H. L. Bowman, S. J. Leonard.

BRANIN, FRANKLIN HULING, South Orange, N. J. (Age 42). Chf. Engr., Structural Steel Board of Trade, Inc., New York City. Refers to W. O. Barkley, J. P. Churchill, D. C. Coyle, A. D. Crossett, R. W. Gastmeyer, G. E. J. Pistor, R. von Fabrice.

BRENNAN, WILLIAM ANDERSON, White Plains, N. Y. (Age 37). Deputy Commr. of Public Safety. Refers to H. C. Atwater, R. E. Dougherty, W. F. Jordan, R. P. Miller, J. H. Myers, A. D. Wolff, Jr., F. C. Zeigler.

BREWER, WILL, St. Louis, Mo. (Age 24). Jun. Engr., U. S. War Dept. Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

BRIDGHAM, MINOT ROBERT SHERMAN, Brooklyn, N. Y. (Age 22). Refers to J. B. Babcock, 3d, H. K. Barrows, C. M. Spofford.

BROOKS, LEONARD, Philadelphia, Pa. (Age 21). Refers to M. Brooks, M O. Fuller, H. G. Payrow, P. M. Sax, C. H. Sutherland.

BROWN, RAYMOND SHEARER, Sharon, Pa. (Age 25) Refers to H. E. Babbitt, H. Cross, J. J. Doland, W. C. Huntington, W. A. Oliver, G. W. Pickels, T. C. Shedd.

BROWN, WALTER AUGUSTUS, Covina, Cal. (Age 23). Refers to E. S. Borgquist, F. C. Kelton, J. C. Park.

BUERKLE, HERBERT COSMOS, Newark, N. J. (Age 27). Asst. Civ. Engr., Eng. Dept., County of Essex. Refers to H. N. Cummings, W. S. LaLonde, Jr.

BURLEIGH, ALBERT FREDERICK, Lincoln, Nebr. (Age 22). Refers to M. I. Evinger, C. E. Mickey.
BURRILL, CECIL LLOYD, Seattle, Wash. (Age 26). Refers to G. E. Hawthorn, C. C. More, R. G. Tyler.
CACACE, LOUIS, ROBERT, Yonkers, N. Y. (Age 23). Refers to J. J. Costa, A. V. Sheridan.
CALMER, JOHN AUGUSTIN, La Canada, Cal. (Age 23). Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.
CAMPBELL, ROBERT LELAND, Jefferson City, Mo. (Age 28). Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.
CANTINE, THOMAS ROBINSON, Portland, Ore. (Age 24). Refers to J. B. Alexander, A. Bauer, G. H. Canfield, J. R. Griffith, D. C. Henny, B. S. Morrow, H. S. Rogers.
CARAMANIAN, ARA, Newark, N. J. (Age 23). Refers to H. N. Cummings, W. S. LaLonde, Jr.
CARLSEN, CHRISTIAN ELMER, Indianapolis, Ind. (Age 22). Refers to W. K. Hatt, S. C. Hollister, R. B. Wiley.
CARRIER, ROBERT TYRRELL, Corfu, N. Y. (Age 24). Refers to G. H. Elbin, A. R. Webb.
CASE, CHARLES ROBERT, Randolph, Ohio (Age 21). Refers to G. H. Elbin, A. R. Webb.
CASEY, JOHN JAMES, San Francisco, Cal. (Age 42). City Engr., City and County of San Francisco. Refers to M. J. Callaghan, A. J. Cleary, L. H. Nishkian, J. M. Owens, F. O. Shutts, H. J. Stahle, L. G. Tegtmeyer.
CASPAR, FERDINAND EDWARD, Orange, N. J. (Age 31). Refers to H. N. Cummings, W. S. LaLonde, Jr.
CLARK, JOSEPH THOMAS, Atlanta, Ga. (Age 26). Refers to J. W. Barnett, R. P. Black, E. N. Seymour, F. C. Snow.
COLICCI, PACIFICO ANTHONY, Providence, R. I. (Age 22). Refers to C. D. Billmyer, J. L. Murray.
COLLINS, ALBERT BERNHARDT, Huntington Park, Cal. (Age 39). Inspector with Los Angeles County Flood Control. Refers to E. C. Eaton, R. M. Fox, N. B. Hodgkinson, F. C. McMillan, B. R. Metcalf, C. E. Pearce, A. E. Sedgwick.
COLLINS, WILLIAM TAFT, Columbus, Ohio (Age 22). Refers to C. T. Morris, R. P. Powell, J. C. Prior, C. E. Sherman.
COOPER, FRANK SCOTT, JR., Roanoke, Va. (Age 23). Refers to G. E. Beggs, F. H. Constant.
COUCHERON-AAMOT, WILHELM, Belleville, N. J. (Age 30). Refers to A. H. Beyer, D. M. Burmister, J. K. Finch, W. J. Krefeld.
CRESSWELL, FREDERICK SORENSEN, Cincinnati, Ohio (Age 24). Inspector and Instrumentman, Cincinnati Union Terminal Co. Refers to H. B. Luther, R. W. Renn.
CROWLEY, LEO FRANCIS, Detroit, Mich. (Age 36). Acting Asst. Engr. of Surveys, City Engr.'s Office. Refers to P. A. Fellows, M. R. Fisher, J. A. Fox, M. F. Wagnitz, F. E. Weber.
CUNNINGHAM, WILLIAM JOHN, Yonkers, N. Y. (Age 22). Refers to W. K. Hatt, W. J. Henderson, S. C. Hollister, F. J. Laverty, G. E. Lommel, G. P. Springer, R. B. Wiley.
CURRAN, CHARLES DANIEL, Vicksburg, Miss. (Age 25). Asst. Director, U. S. Waterways Experiment Station. Refers to L. Brown, E. N. Burrows, J. E. Perry.
CURRAN, DANIEL EDWARD, LaGrande, Ore. (Age 23). Refers to S. M. P. Dolan, J. R. Griffith, G. W. Holcomb, H. S. Rogers.
DARNELL, WILLIAM EDWARD, JR. (Age 21). Refers to C. E. S. Bardsley, J. B. Butler, E. W. Carlton, E. G. Harris.
DEKER, FRANK GEORGE, Philadelphia, Pa. (Age 40). Pres., Cruse-Kemper Co., Ambler, Pa. Refers to H. C. Berry, F. O. Dufour, V. R. Dunlap, S. E. Fairchild, Jr., J. A. Russell, E. R. Schofield, C. L. Warwick.
DENNIS, HAROLD DONALD, Seattle, Wash. (Age 23). Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.
DeRISO, WALTER CARL, North Bergen, N. J. (Age 25). Refers to E. R. Cary, L. W. Clark.
DeWITTE, THEODORE RICHARD, Portland, Ore. (Age 25). Refers to S. M. P. Dolan, J. R. Griffith, H. S. Rogers.
DICKMAN, W. BERNARD, Chicago, Ill. (Age 23). Refers to H. E. Babbitt, H. Cross, J. J. Doland, H. H. Jordan, J. Vawter, C. C. Wiley.
DIGGES, EDWARD WILLIAM, Clewiston, Fla. (Age 37). Asst. Engr., U. S. Army Engrs. Refers to R. B. H. Begg, H. A. Bestor, C. R. Bloxton, R. L. Burwell, E. Mead, R. Y. Patterson, R. W. Wardwell, L. R. Young.
DINKJIAN, NUBAR, West New York, N. J. (Age 22). Refers to F. E. Foss, J. P. J. Williams.
DIXON, JOSEPH GRUNDY, Philadelphia, Pa. (Age 21). Refers to H. L. Bowman, S. J. Leonard.
DORFMAN, JULIUS, Philadelphia, Pa. (Age 24). Refers to H. L. Bowman, S. J. Leonard.
DOUBITZKY, AARON, New York City (Age 26). Refers to H. P. Hammond, E. J. Squire.
DOUGHERTY, DONALD FIX, Dallastown, Pa. (Age 23). Refers to G. H. Elbin, A. R. Webb.
DRAGO, EMANUEL ANTHONY, Mariners Harbor, N. Y. (Age 21). Refers to F. E. Foss, G. Morrison, J. P. J. Williams.
DUYM, WILLIAM ANTHONY, South Orange, N. J. (Age 27). Clerk and Eng. Asst., Bldg. Div., New Jersey Bell Telephone Co. Refers to H. N. Cummings, A. F. Eschenfelder, H. W. Heilmann, W. S. LaLonde, Jr., I. T. Redfern.
EHRLICH, OSCAR CHASKELL, Brooklyn, N. Y. (Age 25). Refers to H. P. Hammond, E. J. Squire.
ELGES, CARL HENRY, JR., Reno, Nev. (Age 22). Refers to F. L. Bixby, H. P. Boardman.
ELLIS, BRUCE WILLIAM, Buffalo, N. Y. (Age 22). Refers to T. R. Lawson, H. O. Sharp.
EVANS, DANIEL, JR., Montclair, N. J. (Age 23). Refers to G. E. Beggs, F. H. Constant.
FABER, BENNIE HERMAN, Austin, Tex. (Age 38). Reclamation Tech. Asst. Engr., Texas Reclamation Dept. Refers to E. N. Gustafson, J. A. Norris, J. J. Richey, A. P. Rollins, G. G. Wickline, B. F. Williams.
FAREGH, MOHAMMAD-ALI SAMIIAN, Cambridge, Mass. (Age 27). Refers to A. Haertlein, L. J. Johnson, C. T. Johnston.
FERRE, HERMAN, Ponce, Porto Rico (Age 23). Asst. Engr., Design Dept., Porto Rico Iron Works, Inc. Refers to J. V. Davila, M. Font, S. Quinones, R. Ramirez, E. Totti y Torres, C. del Valle Zeno.
FIDLER, HAROLD ALVIN, Philadelphia, Pa. (Age 22). Refers to H. L. Bowman, S. J. Leonard.
FISH, FRANKLIN WAKEFIELD, JR., Tucson, Ariz. (Age 23). Refers to E. S. Borgquist, F. C. Kelton, J. C. Park, G. E. P. Smith.
FLORAS, CHRISTOS LAZARE, Athens, Greece (Age 26). Engr., Eng. Div., Ministry of Hygiene, Athens, Greece. Refers to H. G. Baity, W. Donaldson, T. F. Hickerson, T. Saville, R. M. Trimble.

FOLEY, WILLIAM EDWARD, Newport, R. I. (Age 25). Refers to J. J. Costa, A. V. Sheridan.

FOSTER, ELMER LAURENCE, Indianapolis, Ind. (Age 21). Refers to W. K. Hatt, S. C. Hollister, R. B. Wiley.

FRENCH, CHARLES HOTTEL, Woodstock, Va. (Age 21). Refers to W. S. Lohr, L. Perry, E. H. Rockwell, G. F. Roehrig, F. W. Slantz.

FROHBOESE, ERNEST WILLIAM, East Orange, N. J. (Age 21). Refers to H. N. Cummings, W. S. LaLonde, Jr.

FROST, EARL THOMPSON, Lakewood, Ohio (Age 38). Asst. Engr., Cuyahoga County Surveyor's Office. Refers to E. C. Blosser, R. C. Chaney, W. H. Evers, F. A. Pease, G. B. Sowers.

FUTRAL, ALLEN ASHLEY, Savannah, Ga. (Age 20). Refers to R. P. Black, W. L. Brown, J. H. Johnston, G. L. Reed, F. C. Snow.

GAMA, GEORGE HARDING, San Francisco, Cal. (Age 28). Draftsman, San Francisco Water Dept. Refers to A. J. Barclay, I. E. Flaa, J. W. Gross, A. F. Harter, H. F. Jerauld, C. A. Lauenstein.

GARING, ATHOL CLYDE, Seattle, Wash. (Age 22). Refers to I. L. Collier, G. E. Hawthorn, C. C. More, R. G. Tyler.

GAUTHIER, RAYMOND EMILE, San Francisco, Cal. (Age 22). Refers to C. Derleth, Jr., B. A. Etcheverry.

GENDRON, ROLAND ARTHUR, Philadelphia, Pa. (Age 24). Refers to H. L. Bowman, S. J. Leonard.

GOEHRING, FRANK ELDON, Yellowstone Park, Wyo. (Age 21). With U. S. Bureau of Public Roads. Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder, W. H. Thoman.

GRIBBIN, JOHN BRESLIN, Trenton, N. J. (Age 26). Eng. Draftsman, New Jersey Traffic Comm. Refers to A. G. Bisset, J. T. Dean, E. W. Denzler, Jr., W. Z. Kline, A. G. Nicolaysen, B. A. Owen, A. Swan, Jr., W. H. Wilson.

GRIFFITH, JOSEPH GORDON, Manchester Center, Vt. (Age 24). Refers to H. Cross, J. J. Doland, T. C. Shedd.

GUSTAFSON, WILFRED FRANK, Austin, Tex. (Age 25). Refers to E. C. H. Bantel, P. M. Ferguson, J. A. Focht, E. N Gustafson, T. U. Taylor.

HAAG, ADOLPH, New York City (Age 33). Refers to R. L. Bertin, W. K. Brownell, F. O. Dufour, J. L. Orr, O. S. Schlich.

HALLVIK, CARL CLIFFORD, Coeur d'Alene, Idaho (Age 25). Refers to I. N. Carter, I. C. Crawford, J. W. Howard.

HAMILTON, JAMES WILLIAM, Omaha, Nebr. (Age 24). Refers to H. J. Kesner, C. E. Mickey.

HANAUER, MONROE HERMAN, Los Angeles, Cal. (Age 46). Contr. Engr., Minneapolis Steel & Machinery Div., Minneapolis-Moline Power Implement Co. Refers to R. A. Badt, T. A. Beyer, J. B. Gilman, J. W. Mair, S. O. Sprager, A. van Rensselaer.

HANNAH, DAVID MacMORRAN, Batavia, N. Y. (Age 23). Refers to L. M. Gram, T. J. Mitchell, R. L. Morrison, W. C. Sadler, C. O. Wisler, J. S. Worley.

HANSMAN, ARTHUR FRANCIS Albany, N. Y. (Age 27). Asst. Engr., Grade 2, Div. of Highways, New York State Dept. of Public Works. Refers to L. Holmes, H. O. Schermerhorn, A. P. Skaer, W. M. Stieve, N. Stone.

HANSON, ARTHUR HENRY, Tacoma, Wash. (Age 23). Refers to H. E. Phelps, M. K. Snyder, J. G. Woodburn.

HARKER, JOSEPH CLYDE, Los Angeles, Cal. (Age 24). Refers to R. M. Fox, D. M. Wilson.

HARRIS, ROY MONTE, Seattle, Wash. (Age 31). Refers to C. W. Harris, G. E. Hawthorn, R. F. Krafft, C. C. More, R. G. Tyler.

HAWKS, RALPH WAITE, Newburgh, N. Y. (Age 21). Refers to W. W. Rousseau, H. O. Sharp.

HAY, FRANCIS HAYNES, Glendale, Cal. (Age 44). Chf. Hydrographer, Los Angeles County Flood Control Dist. Refers to E. A. Bayley, G. L. Davenport, Jr., J. H. Dockweiler, E. C. Eaton, B. Harmon, H. E. Hedger, L. C. Hill, F. H. Joyner, C. T. Leeds, F. Thomas.

HEBERT, DONALD JOSEPH, Pittsfield, Mass. (Age 22). Refers to C. T. Johnston, H. W. King.

HECTOR, HARTLEY HUMMEL, San Francisco, Cal. (Age 25). Asst. Field Engr., Recreation Comm., City and County of San Francisco. Refers to F. S. Foote, G. S. Harman, P. A. Swafford.

HERBERGER, ARTHUR HENRY, New York City (Age 22). Refers to H. E. Breed, A. Haring, E. G. Hooper, C. T. Schwarze, D. S. Trowbridge.

HIGGS, GEORGE, JR., Astoria, N. Y. (Age 26). Student in Civ. Eng., New York Univ. Refers to W. H. Correale, L. Nadel, C. T. Schwarze, H. H. Snyder, D. S. Trowbridge.

HILL, ROBERT FARRIS, Dalton, Ga. (Age 22). Refers to R. P. Black, W. L. Brown, F. C. Snow.

HILL, WILLIAM CRAWFORD, Minneapolis, Minn. (Age 27). Materials Inspector, Minnesota State Highway Dept. Refers to F. Bass, J. A. Childs, A. S. Cutler, H. M. Hill.

HINCHMAN, JAMES BENJAMIN, St. Louis, Mo. (Age 22). Refers to C. E. S. Bardsley, H. C Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

HOBBS, WILLIAM, JR., Union Bridge, Md. (Age 23). Refers to J. H. Gregory, J. T. Thompson.

HODGES, GLENN EDWARD, Adair, Ill. (Age 23). Refers to J. J. Doland, W. C. Huntington, T. C. Shedd, F. W. Stubbs, Jr., C. C. Wiley.

HODGES, THOMAS LAWRENCE, Swannanoa, N. C. (Age 24). Refers to C. L. Mann, H Tucker, J. S. Whitener.

HOGAN, ELMER ROBERT, Seattle, Wash. (Age 24). Refers to C. W. Harris, G. E. Hawthorn, C. C. May, J. W. Miller, C. C. More, R. G. Tyler.

HOGARTH, CHARLES PINCKNEY, JR., Brunson, S. C. (Age 20). Refers to E. L. Clarke, H. E. Glenn.

HOPWOOD, ROBERT HENRY, Nashville, Tenn. (Age 25). Refers to E. D. Roberts, C. S. Whitney.

HUEBNER, CARL HERMAN, Newark, N. J. (Age 20). Refers to H. N. Cummings, W. S. LaLonde, Jr

HUNT, LOREN WILSON, Berkeley, Cal. (Age 25). Laboratory Asst., Eng. Materials Testing Laboratory, Univ. of California. Refers to R. E. Davis, B. A. Etcheverry, F. C. Herrmann, M. M. O'Shaughnessy, C. R. Rankin, G. E. Troxell.

HUNT, OLIVER PARKS, Green Island, N. Y. (Age 22). Refers to C. E. S. Bardsley, J. B. Butler, E. W. Carlton, E. G. Harris.

HURTGEN, HAROLD RAY, St. Louis, Mo. (Age 23). Refers to M. H. Doyne, R. Sailer, E. O. Sweetser, J. L. Van Ornum.

HYDE, WALTER WILLIAM, Post, Tex. (Age 46). Asst. Res. Engr., Texas State Highway Dept. Refers to C. S. Henning, Jr., G. R. Johnston, T. J. Palm, R. J. Potts, J. G. Rollins.

JANSEN, ALBERT FREDERICK WALTER, Brooklyn, N. Y. (Age 22). Engr., Capitol Steel Corporation of New York. Refers to H. P. Hammond, E. J. Squire.

JENS, STIFEL WILLIAM, University City, Mo. (Age 30). Refers to E. O. Sweetser, J. L. Van Ornum.
JESMAIN, BURT GARDNER, JR., Schenectady, N. Y. (Age 22). Refers to W. W. Rousseau, H. O. Sharp.
JOHNSON, BARCLAY GIDDINGS, Scarsdale, N. Y. (Age 23). Refers to G. E. Beggs, F. H. Constant.
JOHNSON, CHARLES HERBERT, Nashville, Tenn. (Age 54). Asst. Chf. Engr. with Nashville, Chattanooga & St. Louis Ry. Refers to J. M. Johnson, C. E. Kauffmann, F. H. McDonald, W. F. Schulz, C. B. Wilson.
JOHNSON, EMIL UNO, Bronx, N. Y. (Age 24). Refers to F. E. Foss, G. Morrison.
JOHNSON, MASON, Corpus Christi, Tex. (Age 36). Maintenance Project Engr., Texas State Highway Dept. Refers to T. W. Bailey, G. Gilchrist, T. E. Huffman, A. J. McKenzie, W. E. Simpson, G. G. Wickline.
JOHNSON, ROY WILLIAM, Seattle, Wash. (Age 22). Refers to I. L. Collier, C. C. May, C. C. More, R. G. Tyler.
JOHNSON, STANLEY LATHROP, Ossining, N. Y. (Age 22). Refers to J. B. Babcock, 3d, C. B. Breed.
JONES, LOUIS EDWARD, Chicago, Ill. (Age 23). Refers to J. B. Babcock, 3d, C. B. Breed, W. M. Fife.
JONES, WILLIAM PERRY, JR., Urbana, Ill. (Age 21). Refers to H. E. Babbitt, E. E. Bauer, J. S. Crandell, J. J. Doland, W. C. Huntington, T. C. Shedd, C. C. Wiley.
JONY, EMERICH, Philadelphia, Pa. (Age 49). Engr., Pennsylvania Dept. of Health. Refers to A. G. Cherry, H. A. Hageman, T. B. Parker, S. G. Roeblad, J. F. Vaughan, D. M. Wood.
JORGENSEN, ROY ERNST, San Francisco, Cal. (Age 24). Jun. Highway Engr., U. S. Bureau of Public Roads. Refers to H. A. Alderton, Jr., R. E. Davis, C. Derleth, Jr., B. A. Etcheverry, C. C. Morris, J. Moskowitz, C. H. Sweetser.
KABRICH, CHARLES EDWARD, Christiansburg, Va. (Age 24). Refers to R. B. H. Begg, F. J. Sette.
KAHN, EMANUEL LINDE, Chicago, Ill. (Age 22). Refers to J. S. Crandell, W. A. Oliver.
KAMPMEIER, ROLAND AUGUST, Cedar Rapids, Iowa (Age 21). Refers to H. A. Davis, R. B. Kittredge, F. T. Mavis, F. A. Nagler, C. C. Williams, D. L. Yarnell.
KARP, JACOB RESNICK, Roosevelt, N. Y. (Age 32). Senior Topographical Draftsman, Long Island State Park Comm. Refers to H. Five, W. K. Koch, E. L. Lavine, P. H. Lovering, J. J. Weinrib.
KAVANAUGH, WILLIAM FRANCIS, Syracuse, N. Y. (Age 35). Deputy City Engr. Refers to S. N. Grimm, C. S. Herrick, G. D. Holmes, L. Mitchell, M. B. Palmer
KEIM, SAMUEL GEORGE, Hammon, Okla. (Age 23). With Oklahoma Highway Department. Refers to E. R. Stapley, G. B. Stone.
KENNEDY, CHARLES THOMAS, Cincinnati, Ohio (Age 35). Pres., The Kennecrete Co., Inc. Refers to R. W. Bame, W. W. Carlton, E. W. Clark, H. J. Gould, H. W. Hanly, E. C. Harding, J. Lichter, H. D. Loring, H. B. Luther, F. W. Morrill, J. S. Raffety, W. V. Schmiedeke, C. M. Spofford, W. B. Ward.
KENNY, FRANCIS JOSEPH, Norwood, Pa. (Age 22). Refers to H. L. Bowman, S. J. Leonard.
KERN, LOUIS JOSEPH, Elmhurst, N. Y. (Age 22). Refers to H. P. Hammond, E. J. Squire.
KES, ANTHONY, JR., Hoboken, N. J. (Age 25). Draftsman and Engr., New York Switch & Crossing Co. Refers to E. G. Hooper, C. T. Schwarze.
KNOTH, FREDERICK CONRAD, Denver, Colo. (Age 30). Refers to E. O. Bergman, R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder, A. W. Simonds, W. H. Thoman.
KOCHTITZKY, OSCAR WILBUR, JR., Mt. Airy, N. C. (Age 20). Refers to H. G. Baity, T. F. Hickerson, T. Saville.
KOEHLER, WALTER HENRY, New York City (Age 37). Civ. Engr., Pan American Petroleum & Transport Co. Refers to F. E. Foss, C. Kirschner, A. M. McKean, J. B. Stobo, J. R. Stubbins, J. J. Weinrib.
KOFOID, ORVILLE, Portland, Ore. (Age 23). Refers to J. R. Griffith, H. S. Rogers.
KOLB, JOHN THOMAS, Pardoe, Pa. (Age 23). Refers to G. H. Elbin, A. R. Webb.
KOLESOFF, SERGE IVAN, Santa Monica, Cal. (Age 34). With City Engr. Refers to P. W. Clancy, R. M. Fox, F. M. Hines, A. Levan, G. S. Tapley, W. F. Way.
KOPERSKI, JOE JOHN, Chicago, Ill. (Age 21). Refers to J. G. Bennett, T. L. Condron, G. W. Hand, W. E. Hart, W. M. Kinney, C. L. Post.
KOWITZ, ARTHUR WILLIAM, Geneseo, Ill. (Age 22). Refers to H. Cross, J. J. Doland, W. C. Huntington, T. C. Shedd.
KOY, JUSTUS JOHN, Houston, Tex. (Age 23). Clerk, Drafting Dept., United Gas System. Refers to J. S. Broyles, L. B. Ryon, Jr., L. V. Uhrig, R. C. S. Watson, W. E. White.
KRAPF, WILLARD PAUL, Wilmington, Del. (Age 22). Refers to H. K. Preston, R. W. Thoroughgood.
KRISHAN, ROBERT FRANK, Philadelphia, Pa. (Age 21). Refers to H. L. Bowman, S. J. Leonard.
KRANASKAS, ANTHONY JUSTIN, Inwood, W. Va. (Age 23). Instrumentman, Virginia State Road Com. Refers to L. V. Carpenter, R. P. Davis.
KRING, CHARLES UDELL, Urbana, Ill. (Age 21). Refers to W. C. Huntington, T. C. Shedd.
LAGAARD, MAURICE BERNHART, Minneapolis, Minn. (Age 40). Chf. Engr., Minneapolis (Minn.) Bridge Co. Refers to F. Bass, A. S. Cutler, O. M. Leland, F. R. McMillan, G. A. Maney, J. I. Parcel.
LAMBRECHT, RICHARD WALDO, Detroit, Mich. (Age 32). Engr., H. G. Christman-Burke Co. Refers to L. M. Gram, J. T. N. Hoyt, C. W. Hubbell, R. L. McNamee, S. D. Porter, E. C. Shoecraft, J. H. Wasson.
LAMOREAUX, RAYMOND, Schenectady, N. Y. (Age 21). Refers to L. W. Clark, H. B. Compton, T. R. Lawson, W. W. Rousseau.
LANDIS, DAY BLISS, West Orange, N. J. (Age 22). Refers to H. N. Cummings, W. S. LaLonde, Jr.
LANE, THOMAS ALPHONSUS, Roslindale, Mass. (Age 25). Refers to C. B. Breed, C. M. Spofford.
LANGFORD, LEONARD LIONEL, New York City (Age 33). Engr., Pacific-Flush-Tank Co. Refers to H. B. Cleveland, F. S. Friel, S. G. Hess, G. W. Knight, G. B. Mebus, G. L. Robinson, F. K. Wing.
LANIGAN, JOHN JOSEPH, New York City (Age 21). Refers to F. E. Foss, G. Morrison, J. P. J. Williams.
LAST, IRVING, Brooklyn, N. Y. (Age 21). Refers to H. P. Hammond, E. J. Squire.
LAVERGNE YORDAN, LUIS, San Juan, Porto Rico (Age 28). Treas., Earl K. Burton, Inc., Engrs. Refers to E. Baez-Rodriguez, R. A. Beer, J. Benitez-Gautier, S. Quinones, R. Ramirez, C. del Valle Zeno.
LEMASSENA, RICHARD WAYNE, Newark, N. J. (Age 23). Refers to H. D. Allen,

H. N. Cummings, F. F. Griswold, W. S. LaLonde, Jr.

LEMMON, ALLEN BOSLEY, 3D, Palo Alto, Cal. (Age 24). Graduate student in Civ. Eng. (Structural), Stanford Univ. Refers to J. C. L. Fish, F. H. Fowler, E. L. Grant, L. B. Reynolds, J. B. Wells.

LEVANTINE, LEO BURTON, New York City (Age 24). Refers to E. G. Hooper, C. T. Schwarze.

LEVY, GEORGE, Brooklyn, N. Y. (Age 26). Refers to H. P. Hammond, L. F. Rader, E. J. Squire.

LEWIS, CHARLES KIMMEL, Santa Ana, Cal. (Age 22). Refers to W. W. Michael, F. Thomas.

LEWIS, GEORGE NEEDHAM, JR., Baltimore, Md. (Age 25). Inspector, Maryland State Roads Comm. Refers to C. B Bryant, H. G. Campbell, J. H. Ensey, C. D. Jones, A. F. Shure, V. B. Siems.

LEWIS, LLOYD HAMLIN, Wilmington, Del. (Age 21). Refers to H. K. Preston, R. W. Thoroughgood.

LIPP, MAURICE CARL, Roswell, N. Mex. (Age 21). Refers to J. L. Burkholder, J. H. Dorroh, R. G. Hosea, H. C. Neuffer, W. B. Ream.

LO PINTO, VICTOR JOSEPH, New York City (Age 23). Refers to J. J. Costa, A. V. Sheridan.

LUNDIUS, ROY HAROLD, St. Louis, Mo. (Age 21). Refers to C. E. S. Bardsley, J. B. Butler, E. W. Carlton, E. G. Harris, C. V. Mann.

LYNN, AARON VERNON, Blandford, Mass. (Age 31). Asst. Engr., Springfield Water-Works. Refers to R. D. Chase, C. M. Everett, H. H. Hatch, A. H. Holt, E. E. Lochridge, J. L. Tighe, W. F. Uhl.

MacMURRAY, JOHN, JR., West Milford, N. J. (Age 29). Refers to H. F. Peckworth, K. G. Smith.

MacPEEK, ARTHUR WILSON, Newark, N. J. (Age 23). Refers to H. N. Cummings, W. S. LaLonde, Jr.

McCASKEY, AMBROSE EVERETT, JR., New Martinsville, W. Va. (Age 22). Refers to L. V. Carpenter, R. P. Davis.

McCREERY, DONALD HULL, Pasadena, Cal. (Age 33). Engr. and Gen. Supt., Richards-Neustadt Constr. Co. Refers to R. J. Hiller, C. T. Leeds, W. Putnam, R. J. Reed, A. G. Roach, I. L. Tyler, C. White.

McGRATH, JAMES JOSEPH, St. Louis, Mo. (Age 23). Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

McKENZIE, NEVILLE PRICE, Birmingham, Ala. (Age 23). Refers to W. C. Cram, Jr., F. H. McDonald.

McLERNON, RONALD HUGH, Syracuse, N. Y. (Age 22). Refers to W. F. Jordan, L. Mitchell, S. D. Sarason.

McQUEEN, JAMES MILTON, JR., Washington, D. C. (Age 24). Eng. Aide, U. S. Dept. of Agriculture, Bureau of Biological Survey. Refers to M. Falco, O. B. French, J. R. Lapham, P. O. Macqueen.

MABBOTT, LYLE WILLARD, Lincoln, Nebr. (Age 25). Refers to C. M. Duff, H. J. Kesner, C. E. Mickey.

MADDOX, EDWARD FINNIN, Corpus Christi, Tex. (Age 40). Res. Engr., Texas Highway Dept. Refers to R. C. Black, G. Gilchrist, T. J. Kelly, C. H. Kendall, J. M. Page, M. C. Welborn, G. G. Wickline.

MAIS, ERNEST NOEL, Kingston, Jamaica (Age 33). Engr. with Mais & Sant, Engrs. and Contrs. Refers to F. L. Bronstorph, H. J. Dignum, A. A. Simms. (Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

MANGOLD, FREDERICK LOUIS CARL, Milwaukee, Wis. (Age 29). Refers to E. D. Roberts, F. W. Ullius.

MANN, JOSEPH, Edgemere, N. Y. (Age 26). Constr. Supt., Presscott-White Corporation. Refers to E. G. Hooper, C. T. Schwarze.

MARK, RICHARD SHENK, Williamsport, Pa. (Age 24). Eng. Asst., Bureau of Eng., Pennsylvania Dept. of Health. Refers to M. J. Barrick, H. E. Moses, W. L. Stevenson.

MARSON, FRANK MILO, New York City (Age 23). Refers to E. G. Hooper, C. T. Schwarze, D. S. Trowbridge.

MATSEK, JOHN, Roselle, N. J. (Age 22). Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

MATTHEWS, JAMES FREDERICK, Brooklyn, N. Y. (Age 39). Cons. Engr. and Appraiser. Refers to H. K. Endemann, P. P. Farley, A. C. Gallagher, A. J. Griffin, F. W. Newton, E. Praeger, J. E. Tonnelier, L. White.

MEADOWS, CLAYTON JAMES, Houston, Tex. (Age 22). Refers to L. B. Ryon, Jr., L. V. Uhrig, W. E. White.

MIELE, PHILIP VICTOR, Orange, N. J. (Age 25). Refers to H. N. Cummings, W. S. LaLonde, Jr.

MINER, HERMAN ERASTUS, Westerly, R. I. (Age 22). Refers to C. D. Billmyer, J. L. Murray.

MITCHELL, HARRINGTON CALKINS, Wilmette, Ill. (Age 26). Refers to J. H. Cissel, R. A. Dodge, E. L. Eriksen, L. M. Gram, W. C. Sadler, J. A. Van den Broek.

MITCHELL, HENRY BAGLEY, Winchester, Mass. (Age 23). Refers to H. K. Barrows, C. M. Spofford.

MONSON, NORE AGATON, Chicago, Ill. (Age 22). Refers to W. C. Huntington, T. C. Shedd.

MOORE, HILTON HUXLEY, Pequannock, N. J. (Age 21). Refers to H. N. Cummings, W. S. LaLonde, Jr.

MORELAND, OLIVER JAMES, Seattle, Wash. (Age 22). Refers to I. L. Collier, G. E. Hawthorn, C. C. May, C. C. More, R. G. Tyler.

MORT, LINWOOD GEORGE, Jewett City, Conn. (Age 23). Refers to E. R. Cary, T. R. Lawson, W. W. Rousseau.

MOSKOWITZ, HARRY, New York City (Age 38). Constr. and Maintenance Dept., Loew's Inc., and Affiliated Companies. Refers to A. J. Bernstein, H. Cash, M. L. Kaufman, S. Negrey, S. Weishoff.

MUTTERER, WILLIAM ELMAR, Ridgefield Park, N. J. (Age 21). Refers to H. N. Cummings, W. S. LaLonde, Jr.

NARIMAN, RUSTUM KAIKHUSHRO, Secunderabad, India (Age 55). Prof. of Eng., Osmania Univ. Refers to J. R. Freeman, J. B. Lippincott, R. Modjeski, O. J. Todd, J. A. L. Waddell, T. R. J. Ward.

NELSON, HARRY LEGRAND, Scarsdale, N. Y. (Age 23). Refers to E. G. Hooper, C. T. Schwarze.

NELSON, JOHN GEOFFREY, Ft. Benning, Ga. (Age 22). Refers to E. R. Cary, L. W. Clark, T. R. Lawson, W. W. Rousseau, H. O. Sharp.

NETLAND, THEODORE JARL BUGGE, Lodi, Cal. (Age 25). Asst. Hydrographer, East Bay Mun. Utility Dist. Refers to C. E. Grunsky, Jr., L. S. Hall, F. W. Hanna, J. S. Longwell, E. L. MacDonald, L. Netland, F. T. Oakley.

NOLTE, CHARLES BEACH, Chicago, Ill. (Age 46). Executive and Eng. Mgr., Robert W. Hunt Co., Engrs. Refers to W. G. Arn, C. R. Harding, E. T. Howson, D. W. McNaugher, F. M. Randlett, L. E. Ritter, D. B. Rush.

OBERMANN, ROBERT FRANK, Garfield, N. J. (Age 25). Refers to H. N. Cummings, P. W. Fraleigh, W. S. LaLonde, Jr., A. Noack.

OKULOW, NICKOLAS, Brooklyn, N. Y. (Age 37). Refers to A. C. Alvarez, R. E. Davis, C. Derleth, Jr., A. A. Eremin, B. M. Shimkin.

OLANDER, RALPH CARL, Yankton, S. Dak. (Age 25). Equipment Attendant, American Telephone & Telegraph Co. Refers to J. M. Brown, E. J. Stocking.

OLISZEWSKI, CASIMIR, Milwaukee, Wis. (Age 24). Student, Wisconsin Telephone Co. Refers to E. D. Roberts, F. W. Ullius.

PACK, JOHN GEORGE, JR., Bridgeport, Conn. (Age 22). Refers to L. W. Clark, H. O. Sharp.

PAJOT, CLAYTON JAMES JOSEPH, Detroit, Mich. (Age 31). Instructor in Eng. Mechanics, Univ. of Detroit. Refers to A. L. Drabkin, P. A. Fellows, D. P. Gilmore, J. T. N. Hoyt, F. H. Nygren.

PALLER, BEN, Chicago, Ill. (Age 21). Refers to H. E. Babbitt, J. J. Doland, W. C. Huntington.

PARKER, ARTHUR WILLIAM, Ft. Sam Houston, Tex. (Age 51). Capt., Q. M. Corps, U. S. Army. Refers to E. P. Arneson, R. L. Brandt, S. F. Crecelius, L. S. Doten, F. E. Giesecke, E. Gillette, W. S. Goodman, C. R. Goodrich, G. H. Guerdrum, H. R. F. Helland, D. Lee, J. T. L. McNew, F. E. Rightor, N. A. Saigh, U. Stephens, J. F. Woodyard, Jr.

PARSONS, PAUL GATES, Alhambra, Cal. (Age 21). Refers to R. R. Martel, W. W. Michael, A. L. Sonderegger, F. Thomas.

PATTON, MAYNARD ADAMS, Kansas City, Mo. (Age 22). Refers to W. E. Hart, A. S. Hathaway, G. A. Maney, C. L. Post.

PEARSON, HAROLD MILLER, Richmond, Cal. (Age 24). Refers to F. L. Bixby, H. P. Boardman.

PECK, ROBERT RIMER, Camden, N. J. (Age 27). Jun. Engr. and Map Draftsman, New Jersey State Highway Dept. Refers to F. C. Claus, J. L. Dodge, M. W. Grimes, E. H. Maier, E. W. Packer, R. C. Scott, J. A. Williams.

PERRY, PAUL CLUTTER, Little River, Kans. (Age 22). Refers to L. E. Conrad, F. W. Epps, F. F. Frazier, M. W. Furr, C. H. Scholer.

PESCE, CARL ANTHONY, Brooklyn, N. Y. (Age 27). Structural Steel Designer, Board of Transportation. Refers to N. D. Brodkin, J. H. Quimby, E. J. Squire, F. Viola, T. F. Weiss.

PICKERING, HAROLD PHILIP, Kincaid, Kans. (Age 26). Refers to R. L. Downing, F. R. Dungan, C. L. Eckel, E. W. Raeder, H. M. Swope.

POLLOCK, HERBERT WILLIAM, Seattle, Wash. (Age 27). Refers to C. W. Harris, G. E. Hawthorn, R. G. Tyler.

POTTER, ROY WARREN, East London, South Africa (Age 50). Chf. Engr. and Roads Inspector, East London Dist. Council. Refers to H. A. Dunlap, C. N. Forrest, E. A. Pratt, N. Shand, J. H. Weller.

POWELL, FAY EDWIN, Balboa Heights, Canal Zone (Age 46). Engr. and Asst. Constr. Q. M., Panama Canal. Refers to H. Burgess, R. C. Jones, J. L. Schley, W. L. Sibert, R. E. Spaulding.

PREZIOSO, GEORGE SILVIO, Ozone Park, N. Y. (Age 25). Refers to H. R. Codwise, H. P. Hammond, E. J. Squire.

PRINCE, ARCHIBALD VAN BADEN, Webster Groves, Mo. (Age 38). With Constr. Dept., Missouri Pacific R. R. Co. Refers to C. A. Bock, E. A. Hadley, J. A. Lahmer, L. T. Maenner, A. O. Ridgway, C. S. Sample, S. L. Wonson.

QUAM, ELMER RAYMOND, Boulder, Colo. (Age 23). Refers to R. L. Downing, C. L. Eckel.

REEDY, OLIVER CALMAR, Denver, Colo. (Age 22). Refers to M. I. Evinger, W. Grant, H. J. Kesner, C. E. Learned, C. E. Mickey, A. E. Palen, O. T. Reedy.

REINDOLLAR, ROBERT MASON, Baltimore, Md. (Age 38). Asst. Chf. Engr., Maryland State Roads Comm. Refers to S. Eckels, J. S. Howard, A. N. Johnson, G. G. Kelcey, R. Lacy, V. M. Peirce, H. G. Shirley, C. M. Upham.

RENTENBACH, THOMAS JOSEPH, Hancock, Mich. (Age 21). Refers to H. B. Pettit, H. C. Polkinghorne.

REUTER, BERNARD VINCENT, Brooklyn, N. Y. (Age 22). Refers to H. P. Hammond, E. J. Squire.

RICE, CLAUDE HAYES, Dover, N. H. (Age 28). Refers to J. B. Babcock, 3d, C. M. Spofford.

RIDDICK, THOMAS MOORE, Gatesville, N. C. (Age 25). Graduate Student in San. Eng., Univ. of North Carolina. Refers to H. G. Baity, T. F. Hickerson, W. M. Piatt, T. Saville, R. M. Trimble.

RIGGIN, ELDRED CARMEAN, Philadelphia, Pa. (Age 22). Refers to H. L. Bowman, S. J. Leonard.

ROBERTS, WALTER FRIEDGEN, Indianapolis, Ind. (Age 28). Refers to W. J. Henderson, S. C. Hollister, G. E. Lommel, R. B. Wiley.

ROBINSON, GEORGE SYDNOR, Saugerties, N. Y. (Age 22). Refers to L. W. Clark, J. F. Loughran, W. W. Michael.

ROESSER, ROBERT JAMES, Buffalo, N. Y. (Age 23). Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

ROGAN, JOHN EDWARD, JR., New Orleans, La. (Age 20). Rodman, 2d N. O. Dist., U. S. River Comm. Refers to D. Derickson, W. B. Gregory.

ROMANIELLO, CARMINE JAMES, Waterbury, Conn. (Age 22). Refers to L. W. Clark, H. O. Sharp.

ROSCHE, ALFRED PAUL, Normal, Ill. (Age 23). Refers to J. J. Doland, W. C. Huntington, C. C. Wiley.

ROSS, HARRY STEGNER, Cincinnati, Ohio (Age 24). Refers to G. P. Springer, R. B. Wiley.

RUCQUOI, LEON GUILLAUME, Brussels, Belgium (Age 32). Executive Director, Ossature Metallique, Centra belge d'Information de l'Acier. Refers to H. K. Barrows, F. H. Frankland, G. E. J. Pistor, C. M. Spofford. (Applies in accordance with Sec. 1, Art. I, of the By-Laws.)

RUDDER, SAMUEL MILLER, Webster Groves, Mo. (Age 41). Div. Engr., Missouri State Highway Dept. Refers to H. Bartholomew, B. L. Brown, A. P. Greensfelder, W. A. Heimbuecher, W. W. Horner, R. W. Jablonsky, J. C. Travilla.

RUDDY, JOHN MICHAEL, Richmond Hill, N. Y. (Age 23). Refers to J. J. Costa, R. Ridgway.

RUNCK, ROY RUDOLPH, New Madrid, Mo. (Age 35). Res. Engr. with Wilbanks & Pierce, Inc. Refers to W. H. Holland, O. V. Hough, J. C. H. Lee, H. W. Nugent, R. C. Pierce, J. R. Wilbanks.

SABATELLI, EMIL ALBERT, Brooklyn, N. Y. (Age 22). Refers to A. H. Beyer, J. K. Finch, W. J. Krefeld, C. R. Wyckoff.

SALISBURY, LLOYD MOSS, New York City (Age 43). With Constr. Dept., Equitable Life Assurance Soc. Refers to C. B. Ferris, L. O. Marden, W. H. Robinson, G. Simpson, I. R. Smith, R. von Fabrice.

SANCHEZ, JUAN HERMINIO, Austin, Tex. (Age 24). Graduate student, Univ. of Texas. Refers to N. W. Dougherty, H. H. Hale.

SANZENBACHER, WILLIAM PHILLIP, Toledo, Ohio (Age 22). Refers to C. K.

Allen, H. Bouchard, G. Champe, C. S. Finkbeiner, C. T. Johnston, H. W. King, H. W. Knox.

SAWTELLE, EGERTON BURPEE, Ardmore, Pa. (Age 28). Refers to H. L. Bowman, S. J. Leonard.

SCHAD, EUGENE JOHN, Waukegan, Ill. (Age 21). Refers to N. D. Morgan, C. E. Palmer.

SCHMIDT, HENRY EDWARD, Portland, Ore. (Age 22). Refers to S. M. P. Dolan, J. R. Griffith, H. S. Rogers.

SCHMIDT, MILTON ELMER, Minneapolis, Minn. (Age. 22). Refers to F. Bass, A. S. Cutler, H. M. Hill, O. M. Leland, G. E. Loughland, J. I. Parcel.

SCHOFIELD, LOUIS, Brooklyn, N. Y. (Age 28). Refers to H. P. Hammond, E. J. Squire.

SCHULEEN, EMIL PHILIP, Pittsburgh, Pa. (Age 34). Asst. Engr., U. S. Engr. Office. Refers to N. W. Bowden, A. Davis, H. A. Hickman, B. J. Lambert, F. A. Nagler, P. A. Perrin, D. L. Yarnell.

SCOTT, HOMER JUDKINS, Des Moines, Iowa (Age 28). Refers to R. A. Caughey, J. S. Dodds, W. L. Foster, A. H. Fuller, W. E. Galligan, R. A. Moyer, L. O. Stewart.

SCOTT, RODNEY JEROME, Eugene, Ore. (Age 24). Refers to S. M. P. Dolan, J. R. Griffith, G. W. Holcomb, H. S. Rogers.

SCOVILL, FRANCIS LEROY, Brooklyn, N. Y. (Age 30). Engr. (Field), Marcus Contr. Co. Refers to A. H. Diamant, C. E. Fraser, R. E. Goodwin, J. P. Hogan, F. O. X. McLoughlin, F. W. Stiefel.

SEABROOK, CHARLES COURTNEY, Bridgeton, N. J. (Age 23). Salesman, Kosters & Co., Nurserymen. Refers to S. A. Becker, M. O. Fuller, F. C. Hitchcock, E. H. Uhler, W. L. Wilson.

SEARS, ABRAM FUNK, Atkinson, Ill. (Age 22). Refers to H. Cross, T. C. Shedd.

SEGHERS, GUY JOSEPH, New Orleans, La. (Age 34). Member of firm, Ricketts-Seghers & Dibdin, Civ. Engrs. and Surveyors. Refers to A. M. N. Blamphin, J. F. Coleman, J. Klorer, S. F. Lewis, O. K. Olsen, A. F. Theard.

SHOEMAKER, THEODORE, Portland, Ore. (Age 32). Pres., Northwest Roads Co. Refers to T. R. Agg, W. C. Caye, Jr., R. D. Gladding, C. R. Gow, R. G. Hicklin, W. J. Kackley, B. P. McWhorter.

SHOWELL, CARTER SEDDON, Salt Lake City, Utah (Age 27). Refers to G. M. Bacon, R. K. Brown, R. B. Ketchum, F. H. Richardson.

SILLIMAN, JULIAN WINTHROP, Palo Alto, Cal. (Age 23). Graduate student, Stanford Univ. Refers to B. A. Etcheverry, F. H. Fowler, E. L. Grant, L. B. Reynolds, J. B. Wells, C. B. Wing.

SIMS, JOHN PETER, New Boston, Pa. (Age 23). Refers to H. B. Shattuck, E. D. Walker.

SKIDMORE, CLAIDE FERMAN, West Mansfield, Ohio (Age 22). Refers to G. H. Elbin, A. R. Webb.

SMITH, ALBERT VERNON, St. Louis, Mo. (Age 22). Refers to C. E. S. Bardsley, H. C. Beckman, J. B. Butler, E. W. Carlton, E. G. Harris.

SMITH, FREDERICK ANDERSON, Albany, N. Y. (Age 20). Refers to L. W. Clark, H. O. Sharp.

SMITH, HERBERT EDWARD, Brooklyn, N. Y. (Age 22). Engr. Asst., Board of Transportation. Refers to R. E. Goodwin, F. O. X. McLoughlin, J. S. Peck, J. C. Rathbun.

SMITH, JACK KERMIT, Higginsville, Mo. (Age 22). Refers to R. P. Black, F. C. Snow.

SMITH, JESS ANDERS, Woodrow, Colo. (Age 22). Refers to R. L. Downing, C. L. Eckel.

SMITH, LEWIS GORDY, Schenectady, N. Y. (Age 23). Refers to R. A. Hall, J. Macdonald, H. Miller, H. A. Schauffler, W. C. Taylor.

SMITH, ROBERT CHALFIN, Portland, Ore. (Age 21). Refers to J. R. Griffith, H. S. Rogers.

SNELLINGS, HERBERT GLEMM, Norfolk, Va. (Age 22). Refers to R. B. H. Begg, F. J. Sette.

SPANGLER, JOSEPH BROWNE, Houston, Tex. (Age 32). Asst. Engr., United Gas System. Refers to J. S. Broyles, R. J. Cummins, V. E. Hamilton, Jr., R. C. Stokes, R. C. S. Watson.

SPEIDEN, EDGAR FRENCH, Morgantown, W. Va. (Age 24). Refers to G. B. Boomsliter, L. V. Carpenter, R. P. Davis, W. S. Downs.

SPELLMAN, ROGER DRISCOLL, Carnegie, Pa. (Age 38). Structural Engr., Rust Eng. Co., Pittsburgh, Pa. Refers to A. L. Drabkin, B. Goldberg, R. P. Johnson, G. F. Pfeiffer, P. L. Spanne, H. D. Stockwell, J. A. Williams.

SPRAGUE, JOHN HANLY CARROLL, Marmet, W. Va. (Age 30). With U. S. Engrs., Charleston, W. Va. Refers to T. F. Boltz, W. M. Dambach, W. M. Hall, W. H. McAlpine, A. H. Wessel.

SPRINGER, JOHN PUGH (formerly Pugh, John Springer), Palo Alto, Cal. (Age 23). Jun. Structural Engr., Veterans' Administration. Refers to R. E. Davis, C. Derleth, Jr., F. S. Foote, C. G. Hyde, B. Jameyson.

STANIUNAS, JOHN FRANCIS, Hudson, Mass. (Age 23). Refers to L. W. Clark, T. R. Lawson.

STIFLER, FELIX ROYSTON, Bel Air, Md. (Age 41). Foreman of Maintenance, U. S. Govt., War Dept. Refers to F. W. Albert, A. H. Hartman, G. J. Requardt, J. A. White, E. B. Whitman.

STILLMAN, ELI HARRY, New York City (Age 34). Engr. and Estimator, Loew's Theatres. Refers to S. L. Becker, A. J. Bernstein, H. Cash, M. L. Kaufman, S. Negrey, S. Weishoff.

STOCKELBERG, JOHN GERALD, Ancon, Canal Zone, Panama (Age 30). With Raymond Concrete Pile Co. Refers to J. J. Collins, R. A. McMenimen, W. V. McMenimen, J. R. Stubbins, H. E. Weatherlow.

STOEFFEL, WILLIAM HENRY, JR., Bronx, N. Y. (Age 26). With Board of Transportation, New York City. Refers to C. F. Dykeman, L. E. Robbe, P. Sander, J. C. Scott, J. M. C. van Hulsteyn.

STOOLE, SAMUEL, Denver, Colo. (Age 21). Refers to C. L. Eckel, E. W. Raeder.

STOUTENBERG, JOHN HENRY, Brooklyn, N. Y. (Age 20). Refers to T. R. Lawson, H. O. Sharp.

STRONG, JAMES HENRY, Cristobal, Canal Zone (Age 36). Asst. Structural Engr., Fleet Air Base, Coco Solo, Canal Zone. Refers to I. L. Collier, C. H. Cotter, C. C. More, B. Moreell, A. Parent.

STROYAN, ROBERT, Milford, Pa. (Age 21). Refers to H. K. Kistler, E. D. Walker.

SUTHERLAND, DONALD CLARENCE, Trinidad, Colo. (Age 22). Refers to R. L. Downing, C. L. Eckel.

SUTLIFF, HAROLD LESTER, Palmer, N. Y. (Age 22). Refers to E. R. Cary, L. W. Clark.

SWIATLOWSKI, JOSEPH JOHN, Three Rivers, Mass. (Age 23). Refers to D. C. Billmyer, J. L. Murray.

SWIECH, PAUL CHARLES, Carnegie, Pa. (Age 24). Refers to F. J. Evans, F. M. McCullough, C. B. Stanton, H. A. Thomas.

THOMAS, ROBERT SCOFIELD, Oakland, Cal. (Age 22). Jun. Engr., U. S. Bureau of Reclamation, Oakland, Cal. Refers to R. E. Davis, E. L. Grant, C. Moser, G. E. Troxell.

THOMPSON, GLEN MAXWELL, Lamoille, Nev. (Age 21). With Water Commr., Winnemucca, Nev. Refers to F. L. Bixby, H. P. Boardman.
THROCKMORTON, JAMES SANSBURY, 3D, Caldwell, N. J. (Age 23). Refers to H. G. Payrow, F. L. Stuart, C. H. Sutherland.
TIFFANY, JOSEPH BENJAMIN, JR., Kansas City, Mo. (Age 23). Refers to J. S. Crandell, J. J. Doland, W. C. Huntington, M. S. Ketchum, T. C. Shedd.
TORSKY, NICHOLAS KAPETON, Harrison, N. J. (Age 41). Graduate student, Rutgers Univ. Refers to E. G. Hooper, H. N. Lendall, F. F. Longley, C. T. Schwarze, D. S. Trowbridge.
TRINCHIERI, ALFREDO CARLO, Panama City, Panama (Age 33). Structural Engr., Villanueva y Tejeira, Archts., Engrs. and Contrs. Refers to H. G. Arango, L. Arosemena, H. J. Eder, E. P. Haw, E. Jaen Guardia, L. B. Moore.
TURNEY, JOHN GARRETT Wharton, Tex. (Age 25). County Engr., Wharton County. Refers to G. Gilchrist, E. N. Gustafson, J. T. L. McNew, J. M. Nagle, J. J. Richey, A. P. Rollins, T. B. Warden, G. G. Wickline, M E. Worrell.
UKER, VERNE WALTER, St. Louis, Mo. (Age 32). Asst. Eng. Mgr., Sales-Eng. Dept., Shell Petroleum Corporation. Refers to T. R. Agg, D. G. Coombs, A. H. Fuller, J. H. Griffith, A. Marston.
URBANTKE, MARVIN HUGO, Houston, Tex. (Age 22). Refers to J. M. Howe, A. J. Wise.
URIBE RESTREPO, FEDERICO, Bugalagrande, Valle, Colombia (Age 22). Refers to H. B. Compton, T. R. Lawson, W. W. Rousseau, H. O. Sharp.
VALLIS, JOHN NICHOLAS, Urbana, Ill. (Age 22). Refers to J. S. Crandell, W. C. Huntington, C. E. Palmer.
VOGEL, HERBERT DAVIS,. Vicksburg, Miss. (Age 31). Asst. to Pres., Mississippi River Comm. Refers to C. Derleth, Jr., W. B. Gregory, C. G. Hyde, T. H. Jackson, F. A. Nagler, P. S. Reinecke, C. H. West.
WAHLBORG, ROBERT LAWRENCE, Seattle, Wash (Age 21). Refers to C W. Harris, C. C. More, R. G. Tyler.
WARD, CHARLES ELVERTON, Great Neck, N. Y. (Age 22). Refers to E. N. Burrows, J. E. Perry, P. H. Underwood.
WARNECKE, ROBERT IRVING, Brooklyn, N. Y. (Age 23). Refers to E. R. Cary, T. R. Lawson, W. W. Rousseau, H. O. Sharp.
WEDDINGTON, CHARLES FOREMAN, Refugio, Tex. (Age 25). Office Engr., Texas State Highway Dept. Refers to P. M. Ferguson, M. Ramsay, T. U. Taylor.
WERNER, WILLIAM MURRAY, Shreveport, La. (Age 37). W. Murray Werner, Contr. and Constr. Refers to C. D. Evans, E. M. Freeman, S. E. Huey, E. S. Huntington, T. E. Leahy.
WESTERBERG, TORGNY JOEL, Chicago, Ill. (Age 22). Refers to T. L. Condron, G. W. Hand, W. M. Kinney, G. L. Opper, J. F. Seifried.
WESTERFELD, STUART CLARENCE, Winnetka, Ill. (Age 23). Refers to J. B. Babcock, 3d, C. B. Breed, C. M. Spofford.
WHITE, HARRY EDWIN, Penrose, Colo. (Age 27). Refers to C. H. Bryson, W. C. Huntington.
WHITE, HOWARD LESLIE, Terre Haute, Ind. (Age 30). Refers to J. T. Hallett, R. E. Hutchins, R. L. McCormick.
WHITSIT, LAWRENCE COUSINS, Highland Park, Mich. (Age 21). Refers to A. J. Decker, W. C. Hoad, R. H. Sherlock, C. O. Wisler.
WILLOUGHBY, ROBERT A., Brookline, N. H. (Age 28). Refers to A. L. Hambrecht, E. L. Knebes, J. P. Schwada, M. W. Torkelson, L. F. Van Hagan.
WILSON, CARL FETZER, Boonton, N. J. (Age 21). Refers to H. N. Cummings, W. S. LaLonde, Jr.
WILSON, MASTON ALLAN, Wyoming, Del. (Age 22). Refers to H. K. Preston, R. W. Thoroughgood.
WILSON, WARREN ELVIN, Ithaca, N. Y. (Age 24). Graduate student, Cornell Univ. Refers to A. S. Blank, C. D. Jensen, E. W. Schoder, F. J. Seery, C. H. Sutherland.
WINEGAR, OSCAR, Brooklyn, N. Y. (Age 22). Refers to L. W. Clark, H. O. Sharp.
WINELAND, JEFF ANDREW, Berkeley, Cal. (Age 27). Jun. Engr., Bureau of Reclamation. Refers to R. E. Davis, R. R. Martel.
XAVIER, JULIO FRANCIS, JR., Bristol, R. I. (Age 23). Refers to C. D. Billmyer, J. L. Murray.
YOUNG, ALBERT MARLVIN, Rapid City, S. Dak. (Age 29). Instrumentman, Pennington County Highway Dept. Refers to E. D. Dake, J. H. Lake.
YOUNG, CHARLES FREDERICK, Quincy, Fla. (Age 43). Refers to R. L. Bannerman, R. P. Boyd, W. M. Jarvis, H. J. Morrison, W. E. Wheat, H. Worley, Jr.
YOUNG, GARRED FRANCIS, New York City (Age 36). Refers to E. Anderberg, R. R. Benedict, R. M. Hodges, P. H. Lovering, W. F. S. Root, B. L. Weiner.
YOUNG, PHILLIP GAFFNEY, Refugio, Tex. (Age 28). City Engr. Refers to C. M. Blucher, S. W. Freese, J. B. Hawley, H. R. F. Helland, C. J. Howard, M. C. Nichols, J. D. Waring, Jr.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

ASHLEY, WELLS HINE, Assoc. M., Chicago, Ill. (Elected Nov. 15, 1926). (Age 36). Asst. Engr. and First Asst. to Engr. of Sewer Design, San. Dist. of Chicago. Refers to F. A. Barnes, O. L. Eltinge, W. L. Havens, L. Pearse, H. P. Ramey, C. L. Walker, L. C. Whittemore.
BOWLER, EDMOND WESLEY, Assoc. M., Durham, N. H. (Elected Oct. 21, 1924.) (Age 40). Prof. and Head, Civ. Eng. Dept., Univ. of New Hampshire. Refers to J. B. Babcock, 3d, H. K. Barrows, C. B. Breed, G. L. Hosmer, J. H. Kimball, G. E. Russell, C. M. Spofford.
BROWN, MORRIS, Assoc. M., Pittsburgh, Pa. (Elected Jan. 19, 1920.) (Age 46). Pres., Keystone Eng. Co., Structural Steel Fabricators and Engrs. Refers to S. R. Bellows, E. L. Durkee, F. H. Finch, C. H. Fowler, W. H. Frick, F. Kubitz, P. R. Northart.
CLARKSON, EDWARD HALE, JR., Assoc. M., Hollywood, Cal. (Elected Oct. 10, 1927.) (Age 38). Jun. Civ. Engr. Bureau of Eng., Los Angeles, Cal. Refers to L. W. Armstrong, M. Butler, J. H. Chase, J. D. Faulkner, J. J. Jessup, H. S. Kleinschmidt, W. T. Knowlton, R. A. Skinner, G. S. Tapley, A. P. von Deesten.
COTTER, CARL HENRY, Assoc. M. Bremerton, Wash. (Elected Junior Nov. 28, 1916; Assoc. M. Feb. 25, 1924.) (Age 40). Lieut. Commander, Civ. Eng. Corps, U. S. Navy. Refers to W. H. Allen, G. S. Burrell, E. R.

Gayler, L. E. Gregory, B. Moreell, A. L. Parsons, N. M. Smith, W. F. Way, W. C. West.

COVERT, NEWELL SERINE, Assoc. M., Cold Spring, N. Y. (Elected July 11, 1927.) (Age 41). Asst. Engr., Transit Comm. Refers to H. J. Alexander, H. Heins, W. C. Lancaster, R. Ridgway, C. R. Weaver, G. C. Whitney.

DAVIS, CALVIN VICTOR, Assoc. M., New York City. (Elected Dec. 14, 1925.) (Age 35). Chf. Designer, Ambursen Constr. Co., Inc. Refers to E. H. Burroughs, Jr., M. N. Clair, M. D. Kolyn, H. M. Nabstedt, F. A. Noetzli, S. W. Stewart, C. P. Williams.

FAHRNEY, PAUL LANIER, Assoc. M., San Francisco, Cal. (Elected Aug. 18, 1930.) (Age 35). Gen. Sales Mgr., American Bitumuls Co. Refers to L. P. Campbell, R. D. Hoyt, C. D. Jones, C. L. McKesson, R. M. Morton, A. R. Norcross.

IVANCHENKO, ANDREI IVANOVICH, Assoc. M., Novocherkassk-Don, U. S. S. R. (Elected Oct. 10, 1927.) (Age 42). Asst. Prof. in Hydraulics, Don Polytechnic Inst. Refers to B. A. Bakhmeteff, J. B. Egiazaroff, N. C. Grover, R. E. Horton, A. V. Karpov, J. A. Walls.

JOYCE, WALTER EDWARD, Assoc. M., Kingston, N. Y. (Elected June 24, 1914.) (Age 44). Pres. and Chf. Engr., W. E. Joyce Co., Inc. Refers to H. C. Baird, M. B. Case, W. G. Grove, H. D. Robinson, D. B. Steinman, J. C. K. Stuart, G. B. Woodruff.

McLOUGHLIN, FREDERIC OZANAM XAVIER, Assoc. M., New York City. (Elected Junior April 1, 1914; Assoc. M. April 19, 1920.) (Age 44). Associate Prof., of Civ. Eng., Coll. of City of New York. Refers to A. H. Blanchard, I. Van A. Huie, H. T. Immerman, R. Ridgway, O. Singstad, J. R. Slattery, D. B. Steinman, L. White.

McPHERSON, ALBERT MORTIMER, Assoc. M., Wichita Falls, Tex. (Elected Jan. 14, 1918.) (Age 48). Cons. Engr. Refers to W. M. Eliot, O. A. Faris, O. N. Floyd, B. B. Hodgman, J. L. Lochridge, M. C. Nichols, R. J. Potts, W. H. Prentice, Jr., R. A. Thompson, F. M. Veatch, N. T. Veatch, Jr., P. A. Welty.

MAY, DONALD CURTIS, Assoc. M., Ann Arbor, Mich. (Elected Junior May 6, 1914; Assoc. M. Jan. 14, 1918.) (Age 43). Member of firm, Ayres, Lewis, Norris & May, Civ., Hydr. and Elec. Engrs. Refers to L. E. Ayres, G. H. Fenkell, L. M. Gram, C. T. Johnston, H. W. King, H. E. Riggs, F. H. Stephenson.

MOORE, FRED LAWRENCE, Assoc. M., East Orange, N. J. (Elected Dec. 4, 1922.) (Age 39). With C. B. Comstock, Engr. and Archt., New York City. Refers to E. R. Bear, C. D. Curtiss, J. W. Ferguson, H. C. Paddock, H. H. Pitcairn, W. F. Reeves, J. C. Riedel.

NEFF, NAT HARRY, Assoc. M., Santa Ana, Cal. (Elected Dec. 14, 1925.) (Age 44). County Engr., Orange County, Cal. Refers to P. Bailey, S. V. Cortelyou, W. W. Hoy, J. B. Lippincott, C. A. Smith, A. L. Sonderegger, C. R. Sumner.

PROUTY, WINFRED LAFAYETTE, Assoc. M., Denver, Colo. (Elected June 1, 1925.) (Age 42). Prouty Bros. Eng. Co., Cons. Engrs. and Industrial Appraisal Co., Valuation Engrs. Refers to G. M. Bull, H. S. Crocker, C. L. Eckel, W. B. Freeman, C. M. Lightburn.

PUGSLEY, EDMOND FOLSOM, Assoc. M., Seattle, Wash. (Elected Dec. 14, 1925.) (Age 48). Hydr. Engr. Refers to J. M. Gilman, L. M. Holt, J. Jacobs, J. L. Lytel, O. A. Piper, J. L. Savage, R. H. Stock.

RINGWOOD, THOMAS EDWARD, Assoc. M., Montauk, N. Y. (Elected Oct. 1, 1928.) (Age 37). Chf. Engr., Montauk Beach Development Corporation. Refers to R. T. Betts, F. J. Biele, B. A. Bowman, W. N. Brown, S. Miller, L. V. Morris, J. W. Ripley, A. A. Robbins, D. B. Steinman.

SCOTT, WARREN JOSEPH, Assoc. M., Hartford, Conn. (Elected June 1, 1925.) (Age 36). Chf. Engr., Connecticut State Dept. of Health. Refers to H. R. Buck, L. M. Fisher, X. H. Goodnough, R. H. Suttie, A. D. Weston.

STANSEL, HORACE SYLVAN, Assoc. M., Ruleville, Miss. (Elected July 6, 1920.) (Age 43). Civ. Engr. Refers to J. S. Allen, H. B. Bushnell, J. H. Dorroh, W. E. Elam, D. M. Forester, T. G. Gladney, L. L. Hidinger, L. W. Mashburn, R. F. Rudolph, W. J. Shackelford.

TORKELSON, FRANCIS ARTHUR, Assoc. M., Wauwatosa, Wis. (Elected Feb. 24, 1931.) (Age 45). City Engr. Refers to R. H. Cahill, J. T. Donaghey, L. M. Hammond, F. Nagler, O. C. Rollman, C. S. Whitney.

TRIANA, JORGE, Assoc. M., Bogota, Colombia. (Elected Junior March 11, 1919; Assoc. M. July 6, 1925.) (Age 38). Prof. of Highway and R. R. Eng., National Univ. Refers to F. Andrade, P. E. Caicedo, J. Fajardo, J. Pena Polo, P. Uribe Gauguin, G. Uribe H.

VAN den BROEK, JOHN ABRAM, Assoc. M., Ann Arbor, Mich. (Elected Oct. 9, 1917.) (Age 47). Prof. in Eng. Mechanics, Univ. of Michigan. Refers to R. A. Dodge, E. L. Eriksen, C. M. Goodrich, W. C. Hoad, H. W. King.

FROM THE GRADE OF JUNIOR

ANDERSON, MARSHALL PATTON, Jun., Oxford, Ala. (Elected Feb. 25, 1924.) (Age 32). Refers to L. V. Branch, K. A. Farrell, A. C. Polk, C. D. Riddle, R. B. Shepard, Jr., L. G. Warren, S. H. Woodard.

ANDERSON, WILLARD ARON, Jun., Washington, D. C. (Elected Oct. 1, 1928.) (Age 32). Asst. Maintenance Engr., Govt. Printing Office. Refers to W. S. Anderson, J. W. Dunham, R. G. Focht, F. C. Hilder, A. W. Santelmann.

ANDREWS, ALFRED STOKES, Jun., Lakewood, Ohio. (Elected Dec. 3, 1928.) (Age 29). Dist. Engr., U. S. Fidelity & Guaranty Co. Refers to L. M. Bobeau, W. H. Cook, E. E. Duff, Jr., C. G. Dunnells, R. F. MacDowell, A. M. Quick, R. K. Reznor, C. B. Stanton.

BAKER, ARVID HARRY, Jun., Belleville, N. J. (Elected Nov. 12, 1928.) (Age 29). Asst. Engr., Research and Tests Sec., Design Div., The Port of New York Authority. Refers to L. W. Clark, A. Dana, W. C. Huntington, R. S. Johnston, T. R. Lawson, H. D. Robinson, D. B. Steinman.

BELOUSEK, FRANK ANTONE, Jun., Serres, Greece. (Elected Nov. 12, 1928.) (Age 30). Div. Engr., John Monks & Sons, Ulen & Co. Refers to A. B. Christensen, R. W. Gausmann, F. C. Hitchcock, R. H. Keays, M. R. Keefe.

COOK, RUDOLPH, Jun., Bronx, N. Y. (Elected Oct. 10, 1927.) (Age 29). Engr.-Inspector, City Aqueduct Dept., Board of Water Supply. Refers to R. W. Armstrong, W. D. Kramer, J. S. MacDonald, M. Maguire, B. Marcus, C. L. Spaulding.

CORNELL, ARTHUR LELAND, JR., Jun., Milwaukee, Wis. (Elected Aug. 29, 1927.) (Age 32). Sales Engr., American Bitumuls Co., San Francisco, Cal. Refers to O. B. Bestor, B. L. Crenshaw, J. J. Gantt, C. W.

Haasis, J. Higgs, Jr., C. L. King, C. A. Knowles, H. E. Michaud, D. Ulrich, H. A. Underwood.

CRENSHAW, ALLEN EHLERS, Jun., San Francisco, Cal. (Elected June 4, 1928.) (Age 32). Refers to H. S. Bonte, W. H. Kirkbride, G. A. Posey, J. L. Stacer, L. W. Stocker, J. O. Wanzer.

DAVIS, FRANCIS MARION, Jun., Paris, Tex. (Elected July 16, 1928.) (Age 28). Asst. Div. Engr., Texas State Highway Dept. Refers to J. H. Brillhart, J. A. Focht, W. A. French, H. S. Kerr, J. T. L. McNew, J. E. Pirie, J. J. Richey.

EICHLER, PHILIP HENRY, JR., Jun., Hempstead, N. Y. (Elected Oct. 12, 1925.) (Age 32). Engr., Goldberger Raabin Co., Subway Contrs. Refers to H. J. Alexander, H. E. Bierschenk, H. K. Endemann, C. A. Hunt, C. U. Powell, R. S. Saunders, D. C. Serber.

FINKE, RALPH WILLIAM, Jun., Olympia, Wash. (Elected March 14, 1927.) (Age 30). Bridge Designer, Dept. of Highways, State of Washington. Refers to O. R. Elwell, W. R. Engstrom, J. Jacobs, C. C. More, R. M. Murray, A. M. Truesdell, M. S. Woodin.

FITCH, JOHN DOUGLASS, Jun., Aurora Hills, Va. (Elected March 15, 1926.) (Age 28). Asst. Engr., Chas. B. Hawley Eng. Corporation. Refers to H. K. Barrows, A. C. Giesecke, F. M. Gunby, C. B. Hawley, L. M. Pharis, C. M. Spofford, W. F. Uhl, D. C. Walser.

FOWLE, ROYAL EDGAR, Jun., Watsonville, Cal. (Elected July 14, 1930.) (Age 32). With Granite Rock Co. Refers to R. L. Anderson, S. A. Chapman, F. J. Converse, W. W. Michael, F. Thomas.

GORDON, BENNETT, TAYLOR, Jun., Chicago, Ill. (Elected June 10, 1929.) (Age 30). Designer, Strauss Eng. Corporation. Refers to J. C. Adams, C. H. Clarahan, Jr., R. S. Eyre, C. E. Paine, R. S. Quick, G. S. Richardson.

HUNNICUTT, JAMES MADISON, Jun., Charlotte, N C (Elected Aug 29, 1927) (Age 30). Contr. Engr., Virginia Bridge & Iron Co. Refers to C. A. Baughman, P. A. Blackwell, J. A. C. Callan, C. W. Humphreys, E. S. Humphreys, C. W. Ogden, A. R. Peyton.

JARRETT, JAMES MAURICE, Jun., La-Fayette, Ga. (Elected July 16, 1928.) (Age 28). Special San Engr., U. S. Public Health Service. Refers to H. G. Baity, M. Knowles, C. L. Mann, H. E. Miller, C. W. Smedberg, H. Tucker, J. S. Whitener.

KAROLAK, ALPHONSE, Jun., West Coxsackie, N. Y. (Elected Oct. 10, 1927.) (Age 32). Asst. Engr., Grade 1, N. Y. State Dept. of Public Works, Albany, N. Y. Refers to T. T. Davey, F. E. Foss, H. Grand, L. Holmes, G. Morrison, H. O. Schermerhorn, W. M. Stieve, J. P. J. Williams.

KELLBERG, FREDERICK WILLIAM, Jun., Oakland, Cal. (Elected Oct. 10, 1927.) (Age 32). Structural Engr., San Francisco, Cal. Refers to G. J. Calder, M. C. Couchot, L. J. Jennings, V. R. Sandner, F. H. Spitzer, F. D. Talbot, K. Theill.

LAND, RICHARD IRVING, Jun., Mamaroneck, N. Y. (Elected June 6, 1927.) (Age 31). Asst. to Purchasing Agt. and Contr. Mgr., Marc Endlitz & Son. Refers to F. A. Barnes, F. A. Ciampa, F. W. Gardiner, H. T. Larsen, G. H. Pegram, L. C. Urquhart, C. L. Walker, D. T. Webster.

MOSELEY, HARRY HEBER, Jun., Cleveland, Ohio. (Elected Nov. 12, 1928.) (Age 28). Asst. Civ. Engr. with George B. Gascoigne. Refers to A. A. Burger, I. N. Clover, G. B. Gascoigne, W. L. Havens, C. T. Morris, J. C. Prior, F. C. Tolles.

PARISI, FRANCIS GENEROUS, Jun., New York City. (Elected June 7, 1926.) (Age 32). Cons. Engr., H. W. Benson, Associates. Refers to C. A. Garfield, R. E. Goodwin, A. G. Hayden, L. G. Holleran, F. O. X. McLoughlin, W. T. Webb.

PATTON, DAVID HAMMOND, Jun., New Orleans, La. (Elected Nov. 14, 1927.) (Age 32). Dist. Sales Representative, Johns Mansville Sales Corporation. Refers to G. Conahey, L. W. Greene, H. M. Lathrop, W. C. Mundt, H. L. Rogers, H. E. Van Ness.

RIVIERE, JAMES ANDREW, Jun., Callahan, Fla. (Elected Oct. 1, 1926.) (Age 27). Project Engr., Florida State Road Dept. Refers to J. H. Dowling, J. A. Hammack, J. A. Long, J. R. Slade, F. C. Snow.

SHERMAN, EDWARD PRESSLEY, Jun., Ft. Thomas, Ky. (Elected July 12, 1926.) (Age 32). Engr. and Salesman, Jones & Laughlin Steel Co., Cincinnati, Ohio. Refers to E. Kitchen, R. O'Donnell, J. E. Root, H. B. Shattuck, E. D. Walker.

SPURR, JEROME LYON, Jun., Enfield, Mass. (Elected Oct. 10, 1927.) (Age 27). Asst. Engr., Metropolitan Dist. Water Supply Comm. Refers to N. L. Hammond, H. H. Hatch, K. R. Kennison, W. W. Peabody, F. E. Winsor.

STAFFORD, JULIAN TATE, Jun., Berkeley, Cal. (Elected Dec. 14, 1925.) (Age 31). Structural Engr. with Henry D. Dewell, Cons. Engr. Refers to E. L. Cope, R. E. Davis, H. D. Dewell, M. J. Hvorslev, H. Schorer.

STEWART, FRANCIS ALEXANDER, Jun., New York City. (Elected July 16, 1928.) (Age 32). Topographical Draftsman, Borough of Richmond. Refers to S. C. Gordon, G. L. Lucas, C. M. Madden, T. B. Oakley, V. H. Reichelt, N. T. F. Stadtfeld.

STRAUS, HERMAN LOUIS, Jun., Chicago, Ill. (Elected Oct. 21, 1924.) (Age 32). With Chicago Bridge & Iron Works. Refers to H. C. Boardman, W. J. Carrel, G. T. Horton, W. A. Newman, C. S. Pillsbury, D. V. Terrell, M. J. Trees.

THACKER, GERALD QUINCY, Jun., Berkeley, Cal. (Elected June 6, 1927.) (Age 28). Engr., Standard Oil of California. Refers to C. Derleth, Jr., J. M. Evans, H. H. Hall, L. F. Krusi, J. B. Wells, S. K. Whipple.

THOMAS, MARK EVERETT, Jun., San Jose, Cal. (Elected Oct. 1, 1926.) (Age 27). Civ. Engr. and Surveyor. Refers to M. H. Antonacci, S. A. Chapman, C. B. Goodwin, C. M. Kerr, S. P. Laverty, C. Moser, L. B. Reynolds.

THOMPSON, SOPHUS, Jun., Dallas, Tex. (Elected July 15, 1929.) (Age 30). Associate Prof. of Civ. Eng., Southern Methodist Univ. Refers to L. H. Dodd, T. C. Forrest, Jr., T. G. MacCarthy, W. H. Meier, W. R. Spencer.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

BOARD OF DIRECTION MEETINGS

October 3–4, 1932:
A Quarterly Meeting will be held at Atlantic City, N. J.

FALL MEETING

October 5, 6, 7, and 8, 1932

ATLANTIC CITY, N. J.

October 5, 1932:
Morning.—Technical Meeting.
Afternoon.—Technical Meeting.
Evening.—Dinner and Entertainment.

October 6, 1932:
Morning.—Technical Division Sessions.
Afternoon.—Luncheon and Bridge for Ladies; Golf, Fishing, Boating, etc., for Members.
Evening.—Reception, Entertainment, and Dance.

October 7, 1932:
Morning.—Technical Division Sessions.
Afternoon.—Golf, Fishing, Boating, etc.

October 8, 1932:
Morning.—Trip to Philadelphia, Pa.; Luncheon and Inspection of Pennsylvania Railroad Improvements and Other Points of Interest.
Afternoon.—Bus Excursion to Valley Forge.

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.